

MUIR WOOD LECTURE 2018

Monitoring and Controlling Ground Behavior at the Source
Recent Applications to Pressurized Tunneling

Edward J. Cording

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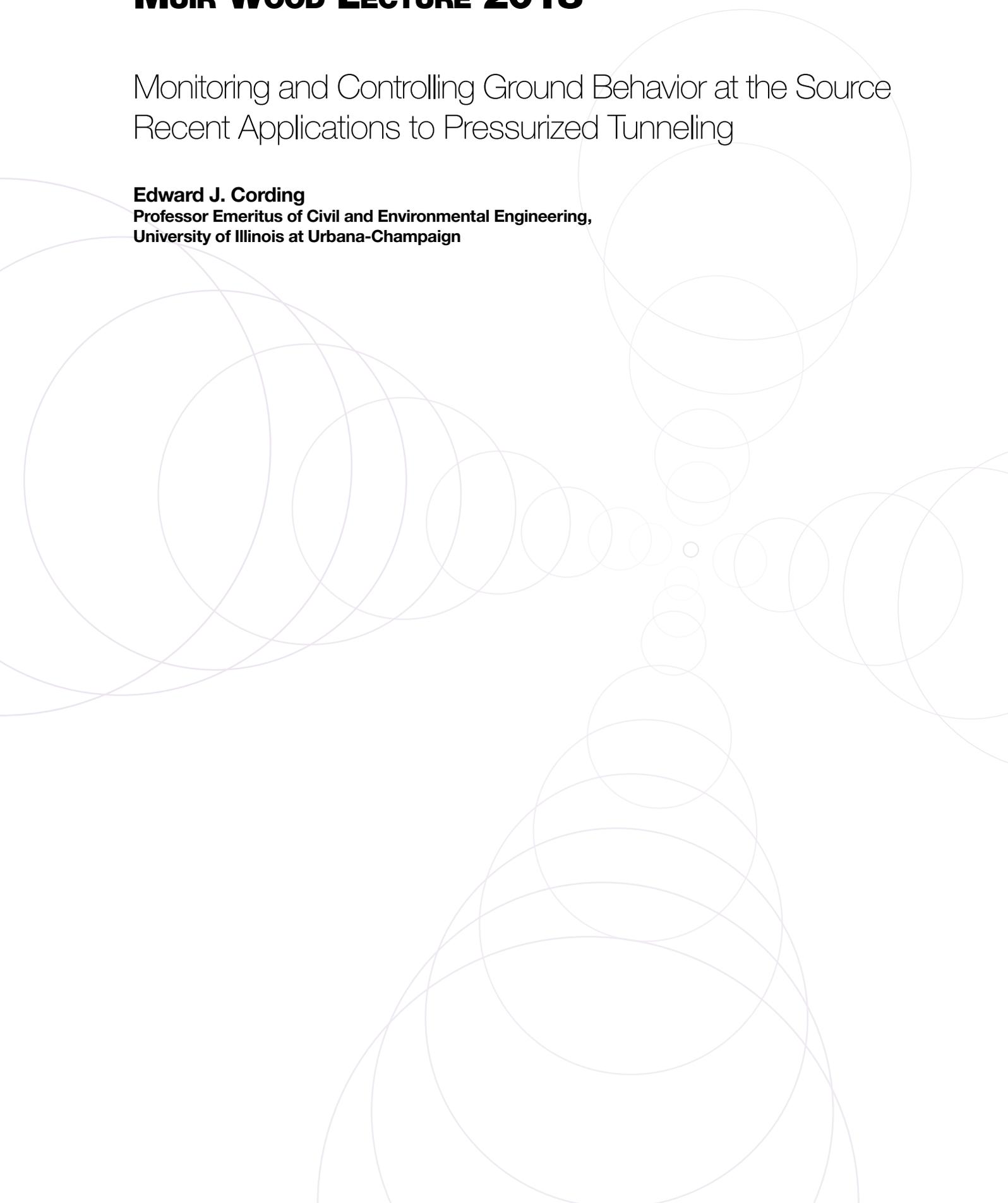
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Recent Applications to Pressurized Tunneling

Edward J. Cording
Professor Emeritus of Civil and Environmental Engineering,
University of Illinois at Urbana-Champaign



>> TABLE OF CONTENTS

ABSTRACT	5
1. INTRODUCTION.....	6
2. SHIELD- DRIVEN TUNNELS.....	7
2.1 THE THAMES TUNNEL 1825 TO 1843: WHERE SHIELD TUNNELING BEGAN	7
2.2 TUNNELING WITH OPEN FACE SHIELDS	7
2.3 GROUND MODIFICATION, REINFORCEMENT, AND REPLACEMENT FOR OPEN SHIELDS AND PRESSURIZED FACE TBMS.....	8
2.4 PIONEERING INVESTIGATIONS CORRELATING THE SOURCES OF GROUND MOVEMENT WITH SURFACE SETTLEMENT, CHICAGO: 1939-1941	9
2.5 GROUND MOVEMENTS DURING SHIELD TUNNELING ON WASHINGTON D.C. METRO: 1970 TO 1974	10
2.6 SHIELD TUNNELING IN THE CHICAGO CLAY: 2000	11
3. PRESSURIZED TUNNELING	13
3.1 PRESSURIZED FACE.....	13
3.2 CONTROL OF REGULAR GROUND LOSS BY FILLING AND PRESSURIZING GAPS	14
3.3 EPBM TUNNELING AT CAPITOL HILL, SOUND TRANSIT U230 PROJECT: 2011	15
3.4 EPBM TUNNELING BENEATH SCHULICH BUILDING, TORONTO: 2012	16
3.5 GROUND CONTROL ON LOS ANGELES METRO TUNNEL PROJECTS	19
4. LARGE DIAMETER PRESSURIZED TBMS AND THE ALASKAN WAY VIADUCT REPLACEMENT TUNNEL.....	21
4.1. INTRODUCTION).....	21
4.2. SITE CONDITIONS ON THE ALASKAN WAY VIADUCT REPLACEMENT PROJECT.....	21
4.3. ASSUMED VOLUME LOSS VS ACTUAL SURFACE SETTLEMENT	22
4.4. INNOVATION AND RISK	23
4.5. COORDINATION PLAN.....	24
4.6. TBM ADVANCE BENEATH ALASKAN WAY VIADUCT AND PIONEER SQUARE	27
4.7. TBM ADVANCE WITH INCREASING DEPTH TO BNSF TUNNEL.....	28
4.8. SETTLEMENT IN SANDS AT NORTH END OF TUNNEL	29
4.9. SETTLEMENT DUE TO DIFFERENTIAL PRESSURE	30
5. CONCLUSIONS.....	31
COORDINATION.....	31
MONITORING GROUND BEHAVIOR AT THE SOURCE	31
PREVENTION OF GROUND LOSS.....	32
6. ACKNOWLEDGEMENTS	33
7. REFERENCES.....	34

Presented are investigations and guidelines for monitoring, and controlling pressurized tunnel boring machines (TBMs) to prevent ground loss, and damaging settlement to structures. The process begins in the planning stages with the owner's team and continues during construction with a coordinated program of managing, operating and monitoring the tunneling process.

Monitoring of ground movements at their source is at the heart of the observational method for geotechnical projects. In large underground rock caverns and on slopes, borehole extensometers and inclinometers have been used for over 60 years to determine the depth of zones of movements and locate the geologic features affecting stability. In urban tunneling and excavation, these instruments are used to locate the sources of ground movement that could cause damaging settlement. For pressurized TBMs, such observations, made close to the advancing TBM, are key to understanding and controlling ground behavior.

Examples are provided of earlier, open face shield tunneling where ground behavior could often be observed directly. The process often relied on the ability of the ground to stand unsupported, and the operator was provided with equipment with which he could not always be successful.

The tunneling industry has witnessed a revolution. Pressurized face TBMs - slurry balance machines and earth pressure balance machines (EPBMs) --- have enabled tunneling at greater depths under waterways and at shallower depths in urban areas without damaging settlement. Advances in recent years have included the increasing capability to coordinate and control the TBM operation and monitor key machine functions and their impact on the surrounding environment in real time as the TBM advances. Maintaining and monitoring a continuously pressurized envelope of conditioned muck and injected slurries around the face and body of the shield, and grout around the lining at the tail of the shield have resulted in improved and consistent control of ground movements throughout the tunnel drive.

In this paper, real time records of key TBM operating parameters are coupled with real time observations of ground movements and porewater pressures immediately around the advancing TBM. Borehole extensometers, piezometers, and directionally drilled horizontal inclinometers are used to pin-point the sources of ground movement and groundwater changes around the TBM to aid in making adjustments and confirming that ground control is being achieved. The results of such observations are described for both open face shields and pressurized face TBMs on tunneling projects in Washington, D.C., Chicago, Toronto, Seattle and Los Angeles.

The benefits have been most dramatic in their application to large diameter TBMs and to TBMs driven at shallow depth in urban areas. The 17.5-m-diameter EPBM selected for the recently-completed tunneling for the Alaskan Way Viaduct Replacement project for State Route 99 in Seattle was the largest pressurized TBM to be driven beneath an urban area. To assess and mitigate risks, the owner and contractor teams built on previous experience with large diameter pressurized face TBMs, including tunnels in Porto (Portugal), Barcelona, and Madrid driven with 9- to 15-m-diameter earth pressure balance machines (EPBMs). Ground improvement and reinforcement methods were implemented, but the primary and most effective ground control measures were in planning and executing the TBM operation. Continuous pressurization of the TBM face, shield steering gap, and lining gap at the shield tail prevented ground loss and prevented damaging settlement throughout the drive beneath Seattle structures.

Essentially, such a tunneling operation achieves the objective described in an 1818 patent application for a tunnel shield by Marc Isambard Brunel of "opening... the ground in such a manner that no more earth shall be displaced than is to be filled by the shell or body of the tunnel" (Muir Wood, 1994, Skempton and Chrimes, 1994). For the 17.5-m EPBM, the settlements immediately above the shield were smallest when the tunnel was at shallow depth, and there was no surface settlement. The results were the opposite of calculations based on an assumed percentage of ground loss, which show the largest settlements and potential damage to structures occur when the tunnel is at shallow depth. That can lead to prescribing ground modification measures or adjusting the alignment to satisfy the assumption, rather than placing primary emphasis on preventing the ground loss by controlling the tunneling operation. Recent progress has made reliable prevention of ground loss possible with the real time linkage of TBM operating parameters and geotechnical observations.

It is of critical importance that there be a coordinated effort among TBM operation and geotechnical monitoring, supervision, engineering, data management, safety, and construction management to assess and mitigate risks, and monitor and control the tunneling operation. Examples of such efforts are provided. Consistently controlling pressures and minimizing settlement throughout the tunnel drive serve as a demonstration to project participants and the community alike that structures along the tunnel alignment will be protected

1 >> INTRODUCTION

Two hundred years ago, Marc Isambard Brunel submitted a patent application, for a tunnel shield with the objective of “opening... the ground in such a manner that no more earth shall be displaced than is to be filled by the shell or body of the tunnel” (Muir Wood, 1994, Skempton and Chrimes, 1994). Efforts to achieve that goal with open-face shields are described in Section 2.

Examples of monitoring and controlling ground behavior at the source include the pioneering observations by Karl Terzaghi and Ralph Peck in 1940 in liner plate tunnels in the soft Chicago Clay. From 1995 to 2000 the Chicago clay was revisited during driving of a 3.6-m open shield with rotating cutterhead. Sources of ground behavior were located using extensometers, inclinometers and piezometers. The results showed that the settlement was primarily caused by loss of ground into the unfilled overcut gap and by time-dependent consolidation of the clay due to the stress changes indicated by the piezometers, conditions that are relevant in pressurized tunneling today.

Current pressurized tunnel boring machines (TBMs), described in Section 3, are capable of controlling not only the stability of the face but also preventing ground loss into the gaps around the shield and tail of the advancing TBM. EPBMs in Toronto, and Los Angeles were advanced at shallow depth in granular soils by filling and pressurizing the gaps around the shield without using the ground replacement procedures such as compensation grouting.

Projects with large diameter TBMs, described in Section 4, of necessity, have led in the development of monitoring and ground control procedures, as shown by EPBM tunneling on Porto Metro, Portugal, and Barcelona's Line 9, and more recently on the 17.5-m-diameter EBPM on the Alaskan Way Viaduct Replacement Project in Seattle. The developments in Porto were two-fold: (1) operation of the TBM included injection of bentonite to positively fill and pressurize the overcut gap around the TBM and injection to maintain pressures in the cutterhead chamber above groundwater levels between advances and (2) development of a coordinated program of managing, operating and monitoring the tunneling process to control ground movements.

On the Alaskan Way Viaduct Replacement tunnel, monitoring of ground behavior at the source was conducted with combination extensometers/piezometers at an average spacing of 16 m, which provided an almost continuous view of the effect of TBM face and shield gap pressures on ground displacement and changes in groundwater pressures. Measurements made in both clays and sands, at depths ranging from 10 to 60 meters, confirmed that the filling and pressurization of the gaps were preventing ground loss. Residual displacements around the advancing TBM were related to the stress changes due to the differential between overburden and face/shield pressures.

Close monitoring of the sources of ground movement on pressurized TBM projects will lead to better understanding of their capabilities and the procedures for consistent operation to minimize risk and prevent damage. Consistently controlling pressures and minimizing settlement throughout the tunnel drive serve as a demonstration to project participants and the community alike that structures along the tunnel alignment will be protected.



2 >> SHIELD-DRIVEN TUNNELS

2.1 THE THAMES TUNNEL 1825 TO 1843 : WHERE SHIELD TUNNELING BEGAN

Marc Isambard Brunel designed, financed, built, and drove the first subaqueous shield under the Thames River, recovered from multiple irruptions of the river into the works, rebuilt the shield under the river, refinanced the project, was knighted in 1841, and finished in 1843. The fascinating story is well known to us because of research and technical papers prepared by Skempton and Chrimes (1994) and by Muir Wood (1994), the title of which is copied as the heading for this paragraph.

Such papers are more than history, they record the engineering/tunneling precedents on which we build. They document the observations of how ground affects tunneling and how tunneling methods control ground behavior. The papers describe how the thinner than expected cover of London Clay led to collapse and flooding of the tunnel. The ground loss leading to this collapse began with flowing on lenses of sand and silt in the tunnel face.

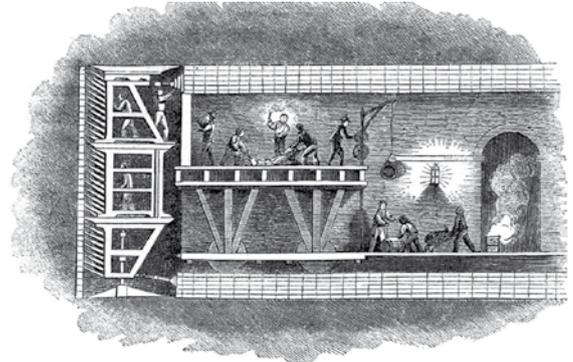
The tunnel was large. It was excavated as a single 12-m-wide box in which a brick-lined double carriage-way was erected under the protection of multiple roof shields, each having three levels with a pocket at the face in which a miner would work, incrementally removing a breast board at a time excavating and setting the board forward a few tens of cm, bracing it with a screw jack against the shield frame, then proceeding downward to the next board (Figure 1a). It was the classic method of breasting in ground with short stand-up time, used in open face shields for the past 175 years.

The tunnel is in use today on the London Underground.

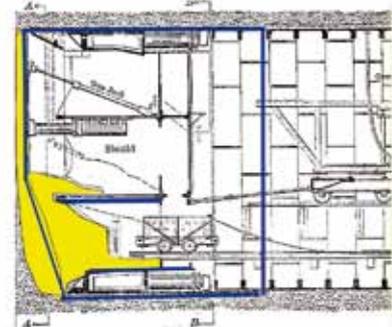
2.2 TUNNELING WITH OPEN FACE SHIELDS

Barlow designed a 2.5-m circular shield, which Greathead drove under the Thames River in 1869. In 1886, Greathead drove twin 3-m-diameter circular shields using compressed air and cast-iron rings that became the standard for open-face hand-mined shields. Open face shields were capable of supporting the face with breast boards supported by screw jacks, and later hydraulic jacks bearing against the frame of the shield so that the shield could be advanced while maintaining support of the breast boards by retracting the jacks (Figure 1c). Some open shields were built with shelves (or tables) so that the excavated soil would form multiple angles of repose to support the face, such as Figure 1b, a compressed air shield under the East River in New York City for Pennsylvania RR. (Noble, 1910, Trans. ASCE).

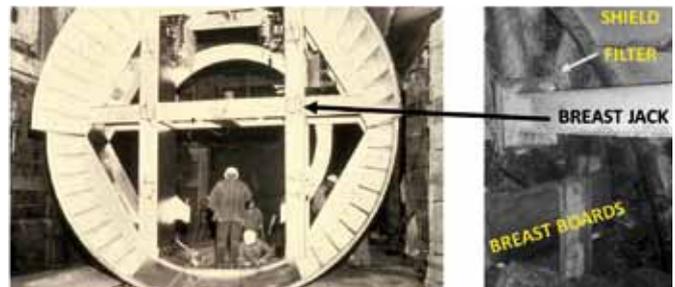
Efforts to improve productivity led to mechanization in the 20th century. Digger shields -- open face shields with mechanical excavators -- typically have a pan in the bottom to allow the muck to form an angle of repose as the shield is pushed forward, thus supporting the lower portion of the face. Figure 1d shows a digger shield with a muck pan designed to provide an angle of repose, and



a. Thames River Tunnel Shield.



b. New York City East River Shield.



c. Open Shield, 1950's.



d. Digger Shield, 1970's.

Figure 1. Open Face Shields.

2 >> SHIELD-DRIVEN TUNNELS

a digger and stinger that could be penetrated into the face to loosen the soils so that the shield could push forward in hard ground and fill the pan. Breast tables and breast flaps were used in the upper face of mechanized shields. Rotating breast flaps such as those shown for the digger shield in Figure 1d were most effective in supporting the upper face during standstill.

Shields with a rotating cutterhead were developed. Lovat shields had guillotine (flood) doors on the cutterhead that could be closed down to prevent inflows and provide access to the muck chamber behind the cutterhead between advances of the shield (Section 2.3)

Often, reliance was placed on the stand-up time of the ground to prevent ground loss into the face of the shields. Stand-up time is reduced when the shield is pushed forward due to vibration and shear forces imposed on the ground.

In flowing, running and raveling ground, the operator could not always be successful in preventing large ground loss with the shield equipment provided. On a project in alluvial sands and clays in Washington, D.C., I observed over two shifts the operation of a shield with a rotating cutterhead being advanced in dry sands. On the first shift, the operator balanced the cutterhead rotation with a steady thrust and shield advance, minimizing ground loss by easing his way through the ground. On the second shift, the operator tended to thrust the rams and crowd the cutterhead, which became torque limited, and then back off on the thrust, rotate the cutterhead with little forward movement of the thrust rams, causing a night of work to fill the void created above the shield.

Urban tunneling with these shields, such as in the alluvial sand, silt and clay on the Washington, D.C. Metro in the 1970s and 1980s, was usually conducted with dewatering wells spaced on 30 - to 50 - m centers, although inflow of perched water on contacts was a common problem and difficult to adequately dewater. Low compressed air pressures (in the range of 0.3 bars to 0.6 bars) were occasionally used.

2.3 GROUND MODIFICATION, REINFORCEMENT, AND REPLACEMENT FOR OPEN SHIELDS AND PRESSURIZED FACE TBMs.

With open shields, in the absence of adequate stand up time in running and raveling soils, it was necessary in a number of cases to use ground improvement or backfilling of voids over significant lengths of the tunnel drive. Several examples follow.

In Phoenix, Arizona, in the 1980s, a digger shield was driven in dry alluvial sand-gravel-cobbles that would stand in the face when the shield was not advancing, but would ravel and run as the shield was pushed forward. Difficulty with penetrating the cobbles to form an angle of repose in the face contributed to the large volume losses and voids that formed above the shield in the first drive, which was largely beneath open fields. Prior to the second drive beneath a street in downtown Phoenix, the decision was made to drill closely spaced holes over the length of the drive, prior to tunneling. A backfilling operation followed

the shield advance, dropping a lean mix of cement and sand to fill the voids as they rose to an overlying caliche layer before they could break through and reach the surface.

In the 1980s, on Sound Transit twin bus/transit tunnels driven in downtown Seattle with digger shields in glacially overridden tills and outwash sands and gravels, ground loss at the face occurred at several locations when the shield encountered raveling and running sands, and was of most concern where the tunnel turned corners and passed beneath buildings. Backfill grout and compaction grout were injected through holed drilled from the surface, from basements, and from the tunnel to fill the voids and compact the loosened zones (Robinson, et. al. (1991).

On two projects driven in alluvial raveling and running sands, one with a digger shield with limited ability to provide an angle of repose in the pan, and the other with a shield with a rotating cutterhead, chemical grout was injected from the surface through a pattern of tube-a-manchette pipes (TAMs) over long reaches of the tunnel to increase stand up time and prevent large ground loss into the tunnel face as the shield passed.

On another project in Los Angeles, where the tunnel turned 90 degrees from one street to another and passed at shallow depth below building spread foundations, chemical grout was injected above the tunnel and below building footings through holes drilled from the tunnel 25 meters forward of the shield.

Extensive permeation grouting or backfilling of voids over long reaches of a tunnel is an obvious condition where the tunneling method does not fit well with the ground conditions.

Pressurized face TBMs have largely eliminated such conditions. Fully pressurized TBMs also prevent ground loss into the gaps around the body of the TBM shield and reduce reliance on permeation grouting or compensation grouting methods that would otherwise be required to prevent structure settlements from exceeding allowable levels. Fully pressurized TBM tunneling, such as cases described in Sections 3 and 4, is capable of controlling settlements to values less than those achievable with compensation grouting.

At the same time, ground improvement measures are an important tool for pressurized TBMs, as well as for sequential excavation methods. Decisions are made to use these additional procedures to reduce risk and maintain stability. Ground improvement procedures are most likely to be used where the TBM cannot be pressurized, for example, at launch and exit when the TBM shield is not fully buried in the ground. They may be used at start-up in the vicinity of critical structures where there has been no monitoring to establish a record of the ground control and there is uncertainty as to the capabilities of the TBM operation to mobilize, pre-check, and test systems before the start. Ground modification may be used in soil or rock with open voids where conditioner and slurries cannot be contained and pressurized.

2 >> SHIELD-DRIVEN TUNNELS

Ground modification procedures can be used to facilitate interventions into the cutterhead chamber for repair and maintenance. Cutterheads may be driven into safe havens constructed of secant walls or jet grout walls for interventions in compressed or free air. Emergency access under free air to repair a damaged cutterhead was achieved on the Port Mann Water tunnel project by ground freezing. The TBM was at a depth of 60 m in granular tills, below the Fraser River in the metropolitan area of Vancouver, British Columbia. The contractor, McNally/Aecon engaged Moretrench, who drilled and conducted the freezing operation from a barge.

There are many other examples. Georgios Anagnostou in his 2014 Muir Wood Lecture “Some Critical Aspects of Subaqueous Tunneling” describes a case on Zurich Cross-rail where large-diameter pipe arches were used to stabilize the ground before driving an 11.3-m-dia slurry shield beneath a structure adjacent to the river, where, because of the shallow, 9-m-depth below the river bottom, pressures required to limit building settlement were close to the total water/soil overburden pressure for the portion of the shield beneath the riverbed. He also describes a large-scale system of porewater pressure relief wells below the seabed on the Storebaelt tunnel to reduce groundwater pressures to levels that could be handled by pressurization of the face.

In urban tunneling, ground control procedures need to extend to the drilling of holes for ground improvement, as well as to the tunneling. It has been observed on several recent projects that the ground movements caused by the controlled pressurized TBM operation were well below those induced by the drilling of multiple holes for ground improvement when drilling techniques were not properly controlled and fitted to the ground conditions. An example where ground movements were controlled during drilling of compensation grout pipes, as well as being controlled during EBPM tunneling, is described in Section 3.5 for the Regional Connector project in Los Angeles.

2.4 PIONEERING INVESTIGATIONS CORRELATING THE SOURCES OF GROUND MOVEMENT WITH SURFACE SETTLEMENT, CHICAGO: 1939-1941

During construction of the Chicago Subway, Karl Terzaghi, Consultant to the City of Chicago, and Ralph Peck, Assistant Subway Engineer, selected by Terzaghi to supervise the soil mechanics laboratory, conducted some of the earliest investigations of the relationship of tunnel construction to surface settlements by measuring the displacement of the clay into the tunnel and observing and recording construction conditions. When the tunnel bottom was in soft clays in the downtown Chicago “Loop”, shields were used with fixed openings at the front through which the clay squeezed (Terzaghi, 1942b). When the bottom of the tunnel was in stiffer clays, to the north of the Chicago River, the liner plate method was used. It was a sequential heading and bench excavation method with compressed air pressures less than 1 bar (Terzaghi, 1942a). A cast concrete lining followed approximately 15 m behind the excavation.

In the sequentially excavated liner plate tunnels, Peck’s team conducted a series of Squeeze Tests over 24- to 48-hour periods, observing and recording the sequence of excavation and support of the heading and bench, sampling soils and measuring the displacement of rods driven into the clay ahead of face and in side walls and arch as the excavation took place.

Surface settlements using the excavation sequence shown in Figure 2 were 100 mm. The Squeeze Tests measurements revealed that much of the ground loss was occurring due to settlement of the steel ribs and liner plate arch as the bench was excavated and posts were placed beneath the arch support.

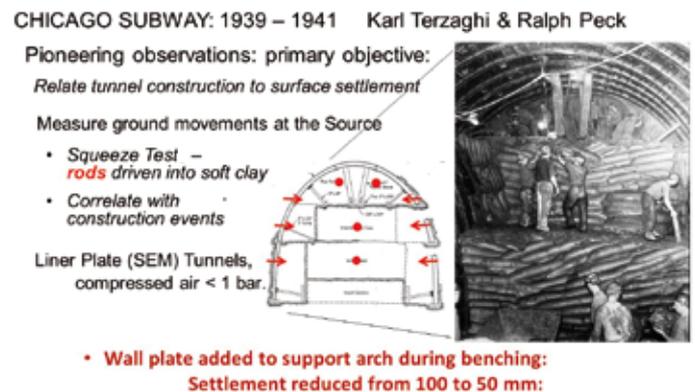


Figure 2 : Chicago Subway: 1939-1941: Observations in the liner plate tunnel.

On subsequent tunnel contracts, surface settlements were reduced to 50 mm by placing wall plate I-beams at the base of the arch to support it longitudinally as the bench was excavated beneath the arch. Two small tunnels (“monkey drifts”) were excavated ahead of the top heading in order to install the I-beams before the arch support was placed (Terzaghi, 1942a)

- On one of the later tunnel contracts, surface settlements again increased to 100 mm as the tunnel depth increased from 12 to 18 m in order to pass below the Chicago River. In one of the early examples of Peck’s use of the observational method, he prepared a Squeeze Test report describing his observations of the construction events affecting the excessive ground loss. (Cording, 2013). Filling of the gap between the clay and liner plate was being delayed several advances behind the top heading excavation. Without any filling, the only support to prevent the clay from squeezing was the compressed air pressure, which could not be increased above the 1 bar maximum to balance the higher overburden stresses. Settlements were reduced from 100 to 50 mm by minimizing and promptly filling the gap when the lining was installed, and by reducing the time and distance of the bench excavations and support installation beneath the top heading.

2 >> SHIELD-DRIVEN TUNNELS



Figure 3 : Filling gaps with pea gravel between the excavated clay surface and the liner plate: Large gaps and delays in filling were a major source of ground loss.

Although the Squeeze Tests were for a sequential excavation method, the approach of assessing the cause of surface settlements by correlating ground loss into the tunnel with construction events was a forerunner of the process of controlling ground movements at their source around pressurized TBMs.

Summary:

- The Squeeze Test was a concentrated period of measuring ground loss at the source, and observing the construction and soil conditions affecting ground loss and surface settlement.
- All the data was summarized on a single blueprint.
- Delayed filling of gaps was a major cause of ground loss and settlement.
- The observations identified the source of ground loss so that construction procedures could be corrected.

2.5 GROUND MOVEMENTS DURING SHIELD TUNNELING ON WASHINGTON D.C. METRO: 1970 TO 1974

The University of Illinois, under contract to Washington Metro, conducted a monitoring program on Phase 1 construction of braced excavations, tunnel and station caverns in rock, and a shield tunnel driven in soil.

For the digger shield tunnel, a test section was established in Lafayette Square, in front of the White House, with multiple position borehole extensometers and inclinometers concentrated around the tunnel. The inclinometer torpedo that was advanced in 0.6-m increments down the casing was equipped with a newly developed servo-accelerometer that could read inclinations to 1/15000, consistent with current capabilities, enabling extension of its use in monitoring slope stability to monitoring

movements around tunnels and excavations. The instrumentation allowed determination of the three-dimensional pattern of ground movements extending from the source of movements around the tunnel shield, distributed through the soil mass to the surface (Cording and Hansmire, 1975, Hansmire and Cording, 1985).

On the first tunnel drive, the level surveys showed large surface settlements of 150 mm. Without the observations of ground movement and the shield operation, there could have been uncertainty as to source, perhaps the assumption that the ground loss was occurring into the tunnel face.

However, the inclinometer located ahead of the face showed only 6 mm of lateral displacement toward the tunnel face (Figure 4).

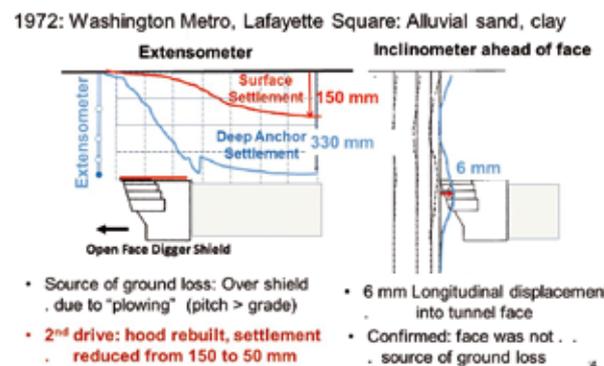


Figure 4 : Ground movements occurring over the shield body of advancing digger shield, Washington, D.C. Metro, 1972.

2 >> SHIELD-DRIVEN TUNNELS

The deep anchor of the borehole extensometers located 0.6 m above the tunnel showed no settlement ahead of the face, but unexpectedly large settlements occurred over the shield, on the order of 50 mm for every 1.2 m advance, for a total of 330 mm.

Measurement with plumb bobs in the tunnel showed that the shield was plowing (pitch of the shield significantly greater than the grade). The shield was not articulated, and had a large hood extending ahead of the face to provide protection during anticipated future hand mining of rock in the invert. In order to maintain the shield on grade, its angle of attack was significantly greater than its grade, causing the front of the shield to be approximately 300 mm above the rear so that progressive settlement occurred over its length.

On the second tunnel drive, the configuration of the hood was changed, reducing the surface settlements from 150 to 50 mm. The volume of the surface settlement trough, as a percentage of the tunnel volume, dropped from 5% to 1.67%. Volume increases occurring above the tunnel caused surface volume to be less than the volume losses around the tunnel.

Ground losses due to plowing and yawing were larger for shields with high length/diameter aspect ratios. In subsequent years, shields were articulated, effectively reducing the aspect ratio, aiding steering and reducing ground loss.

Summary:

- Three-dimensional pattern of vertical and lateral movements in the ground mass and at the surface was obtained with a concentration of extensometers and inclinometers around the tunnel shield.
- Ground movements monitored at the source around the advancing shield, using borehole extensometers and inclinometers, were correlated with shield configuration and operation.
- Cause of ground loss was located and corrected.

2.6 SHIELD TUNNELING IN THE CHICAGO CLAY: 2000

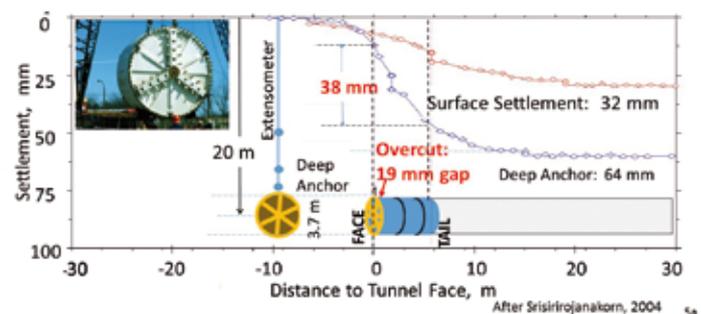
Before the development of pressurized TBMs, ground was consistently lost into the overcut gap (the radial gap between the gauge cutters on the cutterhead and the body of the shield). The gap is required to facilitate steering and reduce friction. The only way to reduce the volume of ground loss was to reduce the size of the gap.

This condition was observed during tunneling in Evanston, Illinois in the soft Chicago clay. McNally Tunneling used a 3.7-m Lovat shield with a rotating cutterhead to drive tunnels on two projects, at depths of 10 and 20 m. The shield had flood (guillotine) doors that could be closed for access to the chamber (Figure 5a). The tunnels connected the Evanston, IL sewer system through drop shafts to the deeper, 10-m-dia storm water storage and transport tunnels located in Silurian Dolomites on Chicago's TARP (Tunnel and Reservoir Project).

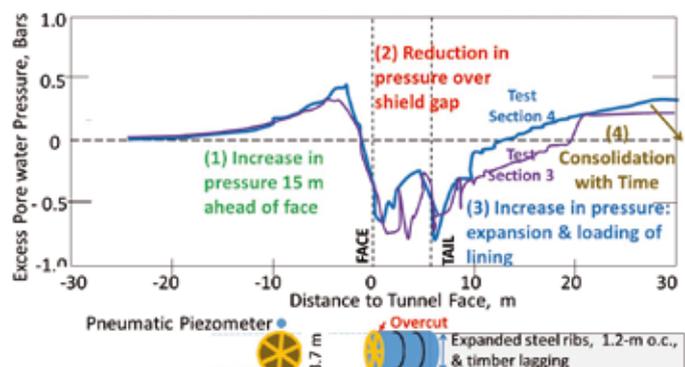
Initial tunnel lining was 100-mm steel ribs and timber lagging, expanded against the ground behind the shield, and final lining was cast-in-place

concrete. The 20-m-deep tunnel was driven beneath a street with adjacent low-rise business and residential structures, and passed beneath commuter rail lines of METRA and Chicago Transit Authority. McNally Tunneling and University of Illinois conducted a joint instrumentation and monitoring program at six test sections along the two alignments, monitoring ground movements and groundwater pressures during and after tunneling (Kawamura and Cording, 1989, Srisirirojanakorn, 2004).

Extensometers located above the tunnel showed that the primary source of the ground loss was the 19-mm overcut gap. Ground loss also occurred prior to fully expanding the steel ribs behind the tail of the shield. At the 20-m-depth, surface settlements after the shield passed were approximately half of the settlement of the deep anchor (Figure 5a). Pneumatic Piezometers and Westbay multiple position piezometers (Figure 5b) recorded the undrained response of the soft clay to the stress changes occurring during tunneling, including (1) the build-up in pressure as the cutterhead approached, (2) the rapid drop in stress over the shield as displacement occurred into the 19 mm radial overcut gap around the shield body followed by a small increase near the back of the shield. (3) Behind the shield, pressures dropped as the ground began to displace into the temporary void created as the steel rib and timber lagging lining emerged from behind the tail and then increased as the lining was expanded against the ground. Pressures continued to increase as the shield advanced and load was transferred to the lining (Figure 5b).



a. Surface and deep anchor settlement, Test Section 4.



b. Piezometric pressures, Test Sections 3 and 4.

Figure 5: Settlements and piezometric pressures during advance of open shield, Chicago Clay: 2000.

2 >> SHIELD-DRIVEN TUNNELS

(4) With time the excess piezometric pressures generated by the stress changes drained back to ambient levels, and additional surface settlement occurred due to consolidation of the lightly over-consolidated clay (Figure 6, TS 4, and Figure 7).

Because the initial lining of steel ribs and timber lagging did not restrict drainage from the clay into the tunnel, a drop below ambient groundwater pressures in the clay resulted in additional consolidation and a settlement of 25 mm (Figure 6, TS 3). The settlement due to drainage was prevented at the crossings beneath the two commuter rail lines by installing a plastic membrane around the steel rib and timber lagging lining as it was erected in the tail of the shield (Figure 6, TS 4). (Gasketed segmental concrete linings will reduce or prevent consolidation due to drainage, depending on the sealing and relative permeability of the lining with respect to the ground.)

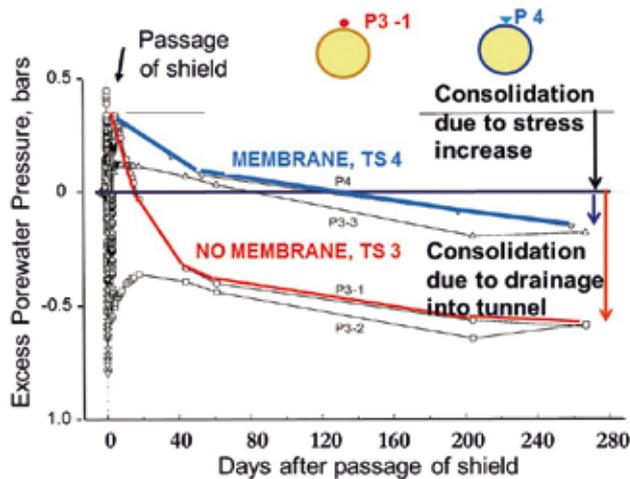


Figure 6: Time dependent changes in excess porewater that caused consolidation of soft Chicago Clay.

Consolidation in TS 4 occurred in a zone around the tunnel, and caused a decrease in volume in the clay and an additional surface settlement of 20 mm. The profile in Figure 7 shows the volume of surface settlement, which is equal to the surface settlement times settlement, trough half width, $w = 2.5 i = (r + z \tan b)$ where i is the point of inflection on a Gaussian curve, r is tunnel radius, and $b = 38$ to 40° , the vertical angle of draw from tunnel springline to the half width of the settlement trough in the clay (Figure 7).

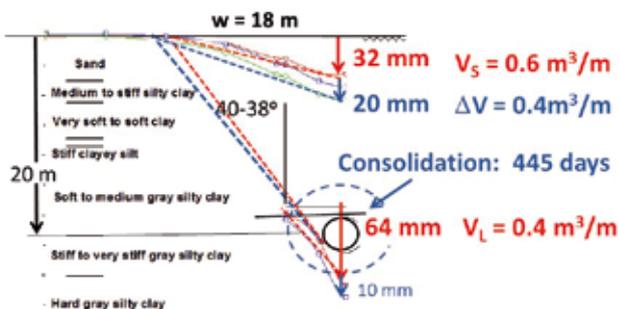


Figure 7: Settlement profile, before and after consolidation of the Chicago Clay, TS-4.

Summary:

- Behavior of the clay with the non-pressurized shield has similarities to a pressurized TBM operation.
- Piezometers record the undrained behavior of clay, and are indicators of the reduction in total stress in the clay due to the presence of the gap.
- Settlements as the shield passed were concentrated above the unfilled shield gap
- Piezometric pressures can be used to monitor pressures and limit consolidation. Pressures above ambient porewater pressures after the shield passed caused consolidation and time-dependent settlement.
- For the permeable lining, additional consolidation occurred due to drainage and drop of porewater pressures below ambient
- Consolidation due to drainage was prevented by installing a membrane around the steel rib and timber lagging. Gasketed segmental linings will reduce or prevent consolidation due to drainage.
- For a pressurized TBM in soft clays pressurization is a balancing act: maintain sufficient pressures at face and in overcut gap to keep the overstress ratio below squeezing $[(YH - P_i) / s_u] < 6]$ where Y is unit weight, H is depth to tunnel crown P_i is face/shield pressure and s_u is undrained shear strength. At the same time, prevent excess porewater pressures after the shield passes in order to minimize settlement due to consolidation.

3 >> PRESSURIZED TUNNELING

3.1 PRESSURIZED FACE

3.1.1 Monitoring Ground loss at the face

In shield tunnels, the potential for over-excavation and large ground loss into the face was checked by measuring volume or weight of muck cars. In EPBMs, the primary means is to measure the weight of the muck on the conveyor belt using two weight scales (Figure 8). The weights are reconciled to the calculated weight of the volume of soil excavated during the advance, with adjustments for the weight of all fluids injected into the face, chamber, and screw conveyor during the advance. Continuous monitoring and display of the volumes with respect to the cutterhead advance has proven to be useful information for the operators.

Monitoring the weight scales throughout the advance is important for checking for large local large ground losses. If there are indications of possible voids, secondary grouting through the segments is used to check for and fill voids. Grout pressures should be high enough and drill set-up on the trailing gear should allow drilling far enough above the segments to intercept rising voids.

Extensometers are used to measure the regular ground losses occurring around the shield but large local ground losses in the face will usually not occur at the extensometer location.

The primary control of ground loss into the face of a pressurized face TBM is proactive control and monitoring of pressures to balance both the dynamic groundwater pressures and the effective active pressure during and between advances of the TBM.

Conditioning of the muck with additives, including foam, polymer and

advances in soils permeable to air, is an important part of controlling the face of an EPBM (Garabagh, et al., 2012). Pressure gradients measured by the earth pressure gauges distributed over the height of the cutterhead chamber are used to identify air bubbles (Mosavat and Mooney, 2005).

Examples of face pressure measurements with time are shown for two tunnel projects (Figures 9 and 10).

3.1.2 Face pressures at York University test section, Toronto: 2012

Figure 9 shows the upper, middle, and lower face pressures (measured by earth pressure gauges mounted on the back wall of the chamber) for an EPBM driven through a 140-m test section in sandy glacial soils in Toronto. Very consistent pressures were maintained throughout the drive of approximately 92 1.5-m advances, which was accomplished in five days, on a 24-hour basis;

The drop in pressures after each advance demonstrates that face pressures were maintained above the static groundwater pressures during the advance. The face pressures were also consistent, without rapid excursions that could cause them to locally drop below the dynamic groundwater pressures around the cutterhead and allow inflow of soil (Figure 9).

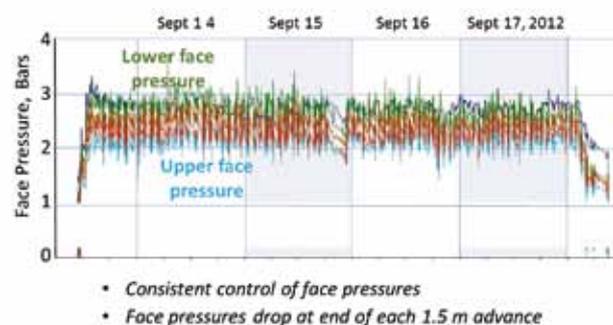


Figure 9 : Face (chamber) pressures during 2nd EPBM drive through test section, York University, Northern Tunnels, Toronto.

3.1.3 Face pressures and ground loss at Beacon Hill tunnel, Seattle

Face pressures rising at the end of the shove provide evidence that they are not high enough to balance the ground water pressures. Figure 10 is a profile of a tunnel section in sandy glacial soils on the Beacon Hill Project, Sound Transit, in Seattle, Washington (Robinson, et. al. 2012, 2013). The plot shows two locations where pressures increased at the end of each 1.5-m advance, indicating that the EPBM was being operated at face pressures below the static groundwater pressure. The conveyor belt weights of the excavated muck at those locations showed high values which were apparently thought to be erratic, however, they were accurate. Both belt weight scales showed values significantly greater than the target weights, which corresponded with the increasing face pressures.

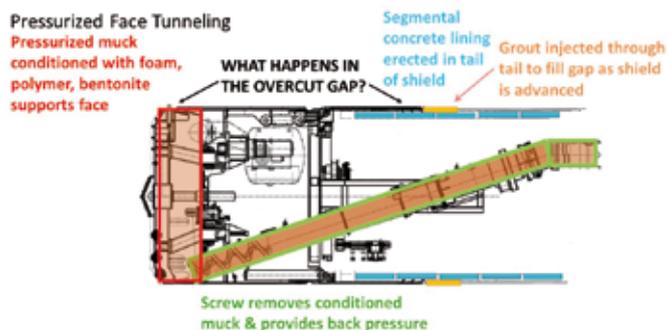


Figure 8 : Earth Pressure Balance Machine (EPBM).

bentonite that fit the soil conditions, is a critical part of an EPBM operation in order to provide constant pressurization of the face throughout the tunnel drive and to make the muck a viscous fluid that will flow through the chamber and screw conveyor, and be transportable on the conveyor belt. Automatic injection of bentonite slurry into the chamber and in the overcut gap around the shield body to maintain pressures between advances is described for the EPBMs in Section 4. Checking for and venting air bubbles that form in the chamber, both during and between

3 >> PRESSURIZED TUNNELING

As a result, sands were flowing into the cutterhead and large volume losses were occurring. The voids migrated upward in the sands to a hard clay till layer above the 40-m-deep EPBM. Months later voids were discovered and holes were drilled from the surface at the locations of the high muck weights to locate and backfill the voids with a low cement content sand mix. Higher pressure compaction grouting was conducted to complete the filling and densification of the voids. Total volume placed was within approximately 2% of the excess volumes calculated from the belt scale weights.

In the central portion of the figure, the upper face pressures balanced groundwater pressures, as indicated by the groundwater pressures dropping slightly at the end of the advance, and the excessive belt weights did not occur.

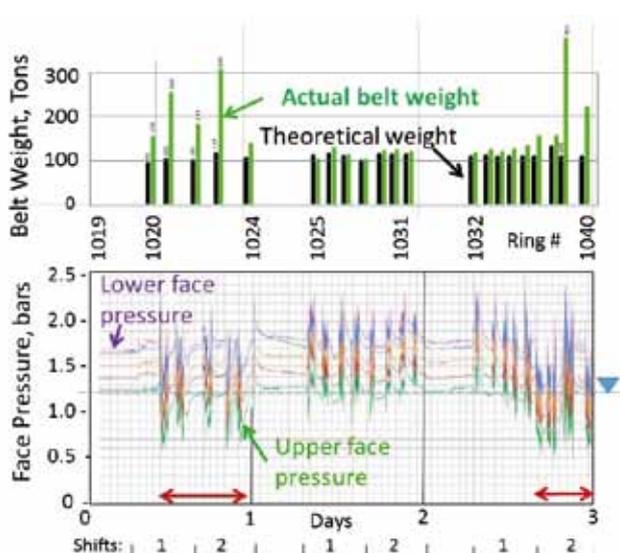


Figure 10 : Beacon Hill Tunnel: Face pressures and large conveyor belt weights.

3.2 CONTROL OF REGULAR GROUND LOSS BY FILLING AND PRESSURIZING GAPS

3.2.1 Potential ground loss around a tunnel shield

Experience with open face shields has shown that the regular ground losses that create the Gaussian-shaped settlement trough can occur (1) at the face, (2) over the shield due to ground loss into the overcut gap or due to plowing or yawing (The advent of articulated shields aided steering and effectively reduced the length/diameter aspect ratio and preventing ground loss due to pitching and yawing of the shield), (3) at the tail during the time taken to grout the gap or expand the lining, and (4) with deflection of the lining.

Pressurized TBMs address all these sources of ground loss.

3.2.1 Pressurizing the face

The primary purpose and benefit of the pressurized face TBM has been to prevent inflows and large ground loss into the face, but pressurized face tunneling also reduces regular ground losses—the smaller elastic displacements at the tunnel face. Face pressures also control the pressures in the shield overcut gap when it is filled, which controls settlement over the shield as described in section 3.2.3.

3.2.2 Filling and pressurizing the tail gap around the tunnel lining

The injection of grout through the tail skin to fill the gap between the perimeter of the shield and the installed lining has become a standard in pressurized tunneling. The gap is large enough, on the order of 150 to 200 mm, that partial filling over several rings would cause very large settlements. The industry has switched from injecting grout through ports in the segments behind the shield to continuous injection of grout through the tail of shield during the advance. A lock-out is used to prevent advance if there are insufficient operational grout ports to fill and pressurize the gap. Two-component grouts with rapid gel time are being widely used. On many projects, including the recent 17.5-m-diameter EPBM in Seattle (Section 4), there was little to say about ground loss into the tail gap, because there was none. Extensometer anchors may show slight upward displacement immediately above the ring being grouted, but the effect is local because the pressures are acting over the narrow 1.2- to 2-m width of the ring. Full grouting around the ring during the advance also prevents squatting of the ring so that there is very little settlement due to lining deflection.

Two-component grouts with rapid gel time have also been specified on recent tunnels driven with non-pressurized TBMs in rock where the concern is to prevent flow of grout forward into the open shield gap. The system was required by Northeast Ohio Regional Sewer District for the Euclid Creek tunnel in Cleveland, Ohio, a 7 m diameter tunnel driven in the Chagrin shale and supported with a segmental concrete lining. McNally-Kiewit, the tunnel contractor, successfully tested and operated the system throughout the tunnel drive. The system provides a more positive and complete filling of the tail gap, replacing the often uncertain and difficult procedures for filling the gap that had previously been used in rock tunnels, such as delayed grouting through the segments with a sloping grout surface behind the shield that leaves the ring initially unsupported, or the multi-step process of injecting pea gravel and then later injecting cement grout into the pea gravel.

3 >> PRESSURIZED TUNNELING

3.2.3 Filling and pressurizing the steering (overcut) gap around the body of the shield

The overcut gap has not received as much attention as the tail gap. Extensometer measurements above open face shields have shown that the overcut gap was often the primary cause of the regular settlements, as illustrated for tunneling in the Chicago clay in Figure 6.

With EPBMs, there is evidence that conditioned muck is more likely to flow into the overcut gap in fine-grained soils than in coarse-grained soils. Extensometer measurements on one EPBM project, without the use of bentonite injection in the overcut gap, showed small settlement in the hard clays throughout the drive but settlement over the shield in the alluvial sands. The configuration at the cutterhead/shield interface and the conditioning of the muck will have an impact on the ability of the muck to flow from the face into the overcut gap.

Loss of ground into an unfilled or partially filled overcut gap will result in large settlements for both shallow tunnels and large-diameter tunnels. Sections 3.4 and 3.5 describe the use of bentonite injection to limit settlement for two EPBM tunnels at shallow depth in sands.

On larger diameter tunnels, increase in the perimeter of the annular gap results in a large increase in its volume. Even if the radial depth of the gap is not increased significantly from that used for smaller diameter tunnels, loss of ground into the gap can result in excessive settlement. Assuming that such ground loss will occur can lead to decisions to increase the depth of the tunnel or provide ground improvement measures. Section 4 describes the first use of bentonite injection in the overcut gap on the Porto, Portugal project, and its use on the 9-m and 12-m EPBMs on Barcelona Line 9, and on the 17.5-m-diameter EPBM in Seattle

3.3 EPBM TUNNELING AT CAPITOL HILL, SOUND TRANSIT U230 PROJECT: 2011

The contractor, Jay Dee, Coluccio, Michels Joint Venture (JCM JV) selected a Hitachi Zosen EPBM and proposed using a large 100-mm radial overcut to reduce wear on hard facing and cutters and reduce interventions for tool changes for the two 1.5-km drives in glacial lacustrine clays and tills with granular soils and boulders on Sound Transit U230 project in Seattle. The gauge cutters creating the 100-mm radial gap were shell cutters that extended beyond the perimeter of the cutterhead body. The large overcut and configuration of the gauge cutters facilitated flow of conditioned muck from the cutterhead into the overcut (DiPonio, et al: 2012).

Sound Transit was concerned for the ability to control ground settlement with such a large overcut. The contractor proposed a test section at start of tunneling to monitor ground movements around the shield with borehole extensometers. Piezometers were also placed in the borehole to monitor dynamic groundwater pressures. Fluid pressure cells were placed on the shield body to monitor the pressure in the overcut for comparison with face and groundwater pressures (Figure 11).

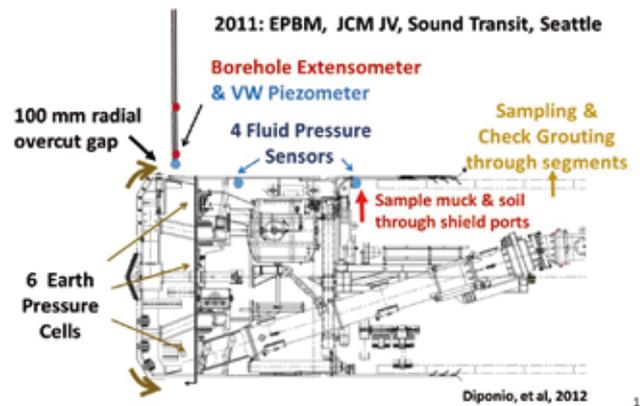


Figure 11 : Monitoring around EPBM, Capitol Hill, Seattle: 2011.

Sampling through ports in the shield showed that the muck was fluid and filling the gap and that the 100-mm gap was being maintained. The instrumentation showed that the fluid pressures in the overcut tracked with the face pressures. Face and shield body pressures were balanced. Piezometric pressures in the sand lenses built up and tracked below the face and shield pressures as the TBM approached and passed. The deep anchors of the extensometers showed no settlement as the TBM passed.

Extensometer E202 was located above the shield body when the advance was stopped and face pressure was intentionally dropped at Ring 49 to check conditions for future cross passage excavation (Figure 12). The extensometer anchor settled 10 mm, indicating that face/shield pressure was holding the gap open and preventing ground loss, but that only a portion of the 100 mm gap was lost.

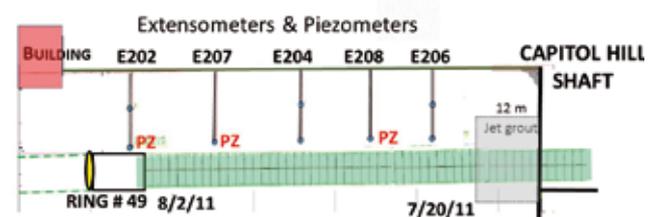


Figure 12 : Extensometers and Piezometers in Test Section, Capitol Hill.

Summary:

- Piezometers were combined with borehole extensometers.
- The large (100-mm) overcut gap and cutterhead configuration facilitated flow of conditioned muck around the cutterhead to fill the overcut gap.
- Fluid pressures in the overcut gap on the shield body tracked with face pressures, and piezometric pressures in the sand lenses built and tracked below the face/shield pressures as the TBM approached and passed.
- Extensometers showed no settlement as the TBM passed.
- Extensometer located above the shield settled 10 mm when face pressure was intentionally dropped.

3 >> PRESSURIZED TUNNELING

3.4 EPBM TUNNELING BENEATH SCHULICH BUILDING, TORONTO: 2012

3.4.1 Plan for controlling settlement using compensation grouting

On the Northern Tunnels Contract for Toronto-York Spadina Subway Extension Project (TYSSE), the contractor was OHL/FCC, and the twin tunnels were driven with two Caterpillar EPBMs (Figure 13). The tunnels were to be driven through the future York University Station site and 6 m below the adjacent Schulich School of Business main building, a prestigious facility on the York University campus consisting of a reinforced concrete frame on spread footings (Figure 14).



Figure 13 : Northern EPBM Tunnels, Toronto.

The special measures developed during design and the measures employed during tunneling are presented in Kramer, et al. 2015. During design, surface settlement volumes were estimated at 0.5% ground loss with a maximum of 1%, which would result in an estimated maximum greenfield settlement of 31 mm (Figure 15).



Figure 14 : York University Schulich Building.

The plan was to perform compensation grouting from three shafts, drilling over 100 tube-à-manchette (TAM) grout pipes to limit the maximum change in vertical building displacements to 15 mm during the sequence of compensation grouting and tunnel advance and to 10 mm after tunneling, thereby reducing damage to very slight or negligible values (Figures 15 and 16).

Imposing the 31 mm ground movement on the structure would result in angular distortion of 3×10^{-3} . Building stiffness reduces the lateral strain and distortion (Figure 16).

3.4.2 Relationship between building distortion and damage

The relationship in Figure 16 represents the state of strain at a point and can be used to represent the average state of strain in the bottom and in the top of a structural bay. The boundary between damage zones is a constant value of the maximum principal tensile strain (Cording et. al. 2001, 2010). Boscardin and Cording (1989) developed the relationship in terms of the maximum angular distortion and lateral strain of a beam with length/height ratio of one, but it is equivalent to the relationship for the state of strain at a point shown in Figure 14, with only slightly different shape of the boundaries for the damage zones. They built on the relationships developed by Burland and Wroth (1974) for bending and shear strains in beams with window penetrations producing high E/G values). The relationship for angular distortions was correlated with the work of Skempton and MacDonald (1956) for settlement of buildings. The lateral strains were correlated with the lateral displacement vs damage criterion developed by the United Kingdom National Coal Board for subsidence over deep mines. Damage level descriptions in the figure were developed for masonry structures by Burland et. al. (1977).

The relationship has general applicability to different types of structures. It is used for bearing walls behaving as a single beam and is also used for assessing average state of strain and damage to the infill or finishes within the bays of a frame structure.

3 >> PRESSURIZED TUNNELING

Stiffness of the ground/structure can reduce both angular distortion and lateral strain estimated from the greenfield settlements. A relationship showing the reduction in lateral strain due to the relative soil/grade beam stiffness is provided in Boscardin and Cording, 1989. Professor Robert Mair, in his 2013 Muir Wood lecture, provides relationships and examples of the effect of building stiffness in reducing settlement and the shape and distortion of the settlement profile.

The tunnel designer, Hatch Mott MacDonald, in assessing the impacts of tunneling on the Schulich building, performed numerical analyses to determine the effect of building stiffness on the distortions using the distortion/damage relationship of Figure 16.

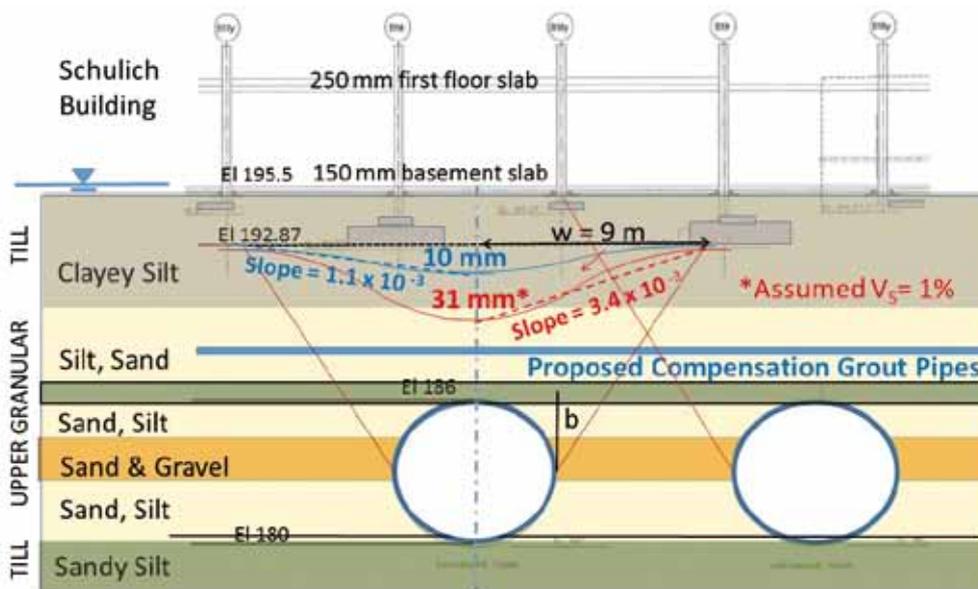
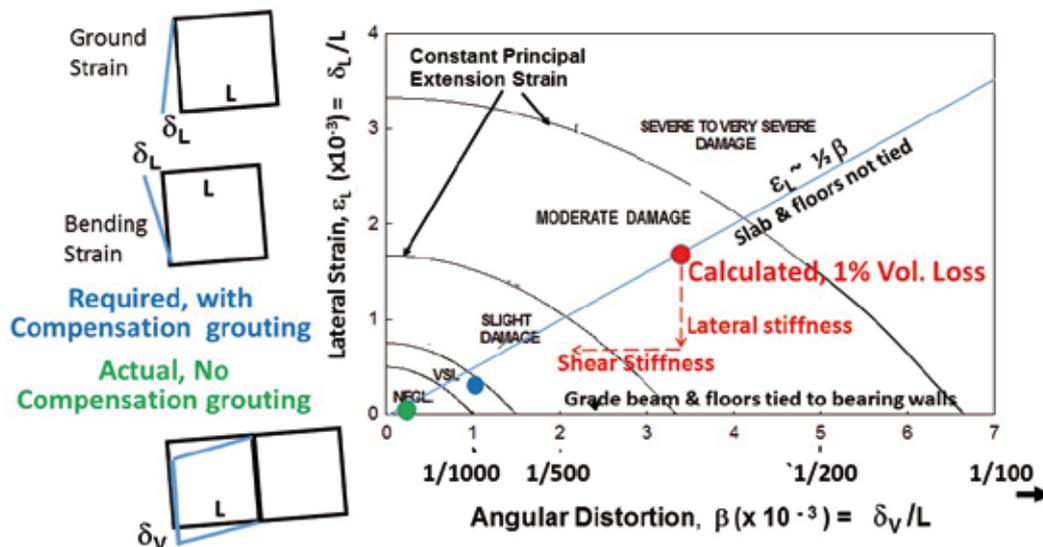


Figure 15 : Profile of ground conditions and predicted settlements at Schulich Building, Northern Tunnels, Toronto.



After Boscardin & Cording, 1989, Cording, et al, 2001

Figure 16 : Damage criterion based on average state of strain in a bay due to lateral strain and angular distortion.

3 >> PRESSURIZED TUNNELING

3.4.2. Test section prior to EPBM tunneling beneath Schulich Building

The three shafts for compensation grouting had been excavated, but the compensation grout holes had not been drilled as the two TBMs reached the future York University Station, 140 m from the adjacent Schulich building. The contractor requested to advance tunnels beneath the building without installing compensation grout pipes.

During the drive of the two EPBMs toward York University, settlement of the surface, 12 m above the tunnels had been small, typically in range of 1 to 5 mm. However, there were limited data from borehole extensometer data showing displacements above the tunnels and at the level of future building foundations the tunnel. Further, there was concern for the ability to control ground movements and fill the overcut gap in the increasingly granular glacial soils at the York University Schulich building.

The Toronto Transit Commission (TTC), and consultants recommended that a test section be installed prior to tunneling beneath the Schulich building to determine if settlement could be consistently controlled within the 10-mm criterion established for the compensation grouting program.

TTC coordinated the test section program and conducted daily reviews with the designer, contractor, construction manager, geotechnical consultant, and instrumentation group. Instrumentation included 33 multiple position borehole extensometers, each with four anchors, 0.7 m above the tunnel and at the level of the future building foundations, as well as precise level surveys of the surface and the extensometer heads, and piezometers. (Kramer, et al, 2015).

The 6.1-m Caterpillar EPBMs had been built with a system for bentonite injection through the shield body but it had not been regularly used by the contractor during the drive toward York University in the predominantly clayey tills. TTC consultants recommended that the system be used throughout the test section and passage beneath the Schulich building to

provide a positive means of filling the overcut gap in the anticipated more granular soils. Fluid pressure cells were also recommended and installed by Caterpillar on the shield body to monitor pressures in the overcut gap.

Static ground water level in the granular soils was 10 m above the tunnel, a pressure of 1 bar on the upper face of the EPBM. On the first drive the upper face pressures were at 1.9 bars and settlements 6 m above the shield were in the range of 1 to 5 mm. On the second drive through the test section, the contractor elected to increase face pressure to 2.2 bars resulting in a reduction of settlements to 1 to 2 mm (Figures 9 and 17).

At the end of the first drive through the test section, upon reaching and embedding the TBM cutterhead in the future station headwall, the face pressures were dropped for maintenance of the cutterhead. Face pressures are coupled with overcut pressures, so that the pressures in the overcut gap also dropped resulting in 13 mm of settlement of the extensometer located 6 m above the shield, a confirmation of the effectiveness of pressurization of the overcut gap in preventing settlement.

Control of the TBM operation by the contractor, OHL/FCC, and the consistency of face pressures, conditioning, and filling of the overcut gap was evident in the two drives through the test section. The settlements of 1 to 2 mm were far below the maximum 10-mm criterion (Figure 17).

The passage beneath the building was accomplished using the same ground control procedures as in the test section. Vertical displacements as measured by liquid level lines, robotic total station measurements and precise level surveys showed a maximum heave of 1.9 mm and settlement of 1 mm over the 81 and 62 ring pushes.

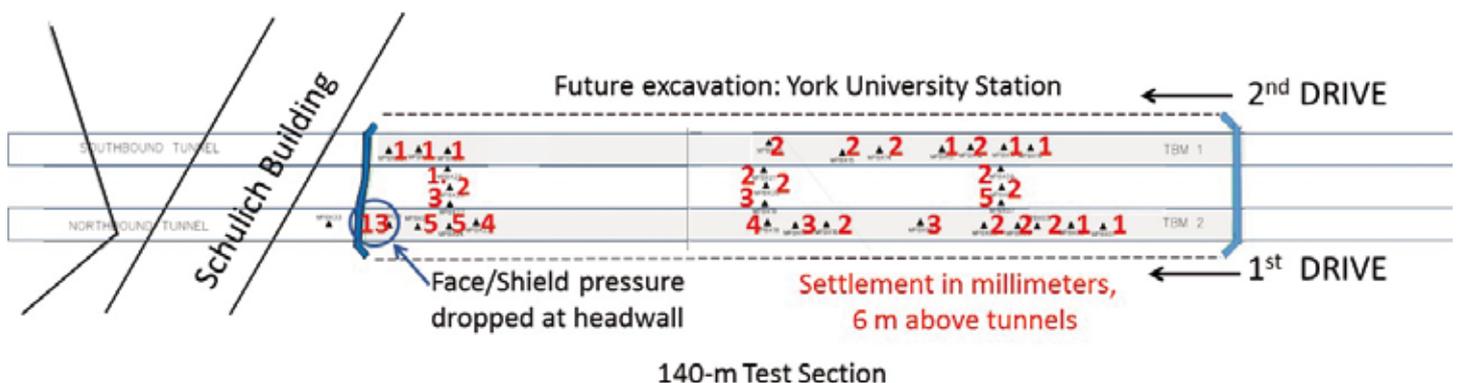


Figure 17 : Settlement 6 m above tunnels after passage of EPBMs through test section, 9/18/2012.

3 >> PRESSURIZED TUNNELING

3.4.3 Summary

- Toronto Transit Commission and contractor, OHL/FCC, conducted a coordinated and cooperative program for monitoring ground behavior and controlling the TBM operation.
- Consistent control of TBM operating parameters, including face pressures, conditioning, and filling of overcut and tail gaps was demonstrated in the 140-m test section.
- Settlements of 2 to 5 mm on the first drive were reduced to 1 to 2 mm on the second drive by increasing face/shield pressures. Settlement was below the 10 mm maximum, and below values achievable with compensation grouting.
- During the drive beneath the Schulich Building settlements ranged from 1.9 mm heave to 1 mm settlement.

3.5 GROUND CONTROL ON LOS ANGELES METRO TUNNEL PROJECTS

3.5.1 Pressurized Tunnel Projects

After LA Metro experience with large ground losses using open face digger shields in sandy alluvial soils in the early 1990s, a Tunnel Advisory Panel was convened in 1994 and concluded that controlled tunneling could be conducted in Los Angeles and that pressurized face TBMs should be utilized.

Gold Line East Side Extension. The next LA Metro tunnel project was the Gold Line East Side Extension, driven in 2006 with two Herrenknecht EPBMs at a depth of 15 to 20 m in Old Alluvium consisting predominantly of silty clays and silty sands. (Choueiry et. al., 2007, Robinson and Bragard, 2007). Compensation grout pipes, although installed beneath structures located near the start of tunneling, were not used during tunneling. Ground settlements throughout the drive were typically in the range of zero to 8 mm. There was no injection of bentonite in the overcut gap in these predominantly dense silty clays and silty sands.

Current Metro projects. On current Metro projects, the maximum surface settlement criterion is 13 mm. A fully pressurized envelope around the TBM face, shield and tail is required, including filling and pressurization of the steering (overcut) gap and monitoring with pressure cells installed on the shield perimeter.

Currently five underground light and heavy rail projects are underway.

Crenshaw. The tunneling portion of the Crenshaw light rail line was completed in 2017. The twin tunnels were driven with a single Herrenknecht EPBM in sand and gravel alluvium, with the tunnel crown above the water table in much of the drive. In these sandy alluvial soils. Once the gaps were filled and ground loss prevented, the remaining settlement was a function of the pressures applied at the face and around the shield body with respect to the overburden stresses.

Regional Connector. TBM tunneling on the Regional Connector light rail line was completed in February, 2018. The first 150 m of the twin tunnel drives was at shallow depth in alluvial sandy soils beneath structures as described in the following sections.

Purple Line. On the Purple Line, which extends west along Wilshire Boulevard through Century City to University of California at Los Angeles and the Veterans Administration Hospital, three heavy-rail sections with twin tunnels and seven station excavations are in design or underway. The Purple Line is predominantly in alluvial soils and in the Fernando formation of very hard silt and clay. Sections of the tunnel pass through soils with high concentrations of methane and H₂S. One section passes through soils infused with naturally occurring asphalt, in the vicinity of the La Brea tar pits.

3.5.2 Regional Connector Project: EPBM start-up at shallow depth beneath structures

On the Regional Connector Light Rail line, the twin tunnels were driven with a single Herrenknecht EPBM. The second tunnel holed through in February 2018, and construction began on sequentially excavated cross-passages and a cross-over cavern. Detailed descriptions of the project and the start of tunneling are provided by Hansmire, et al., 2017.

The first 150 m of the tunnel drive beyond the braced excavation launch shaft passed beneath a reinforced concrete frame parking structure on spread footings and then shops and a mall at a depth increasing from 6 m to 12 m in medium to medium dense clean uniform sand interbedded with silty sand, gravel and well-graded sand. Near the end of the 150-m section, the Fernando Formation, a weak siltstone/claystone (very hard silty clay), occupied most of the tunnel face, with the granular alluvium remaining at and above the tunnel crown.

The contractor, Regional Connector Constructors (RCC), drove the twin tunnels with a rebuilt Herrenknecht EPBM that had been used to drive one of the twin tunnels on the LA Metro Gold Line East Side Extension in 2006 and one of the twin tunnels on Seattle's Sound Transit U220 University Link project in 2011. For the Regional Connector project, a system was added for filling and pressurizing of the steering (overcut) gap, monitored with six pressure cells on the shield perimeter.

Contract documents required installation of a compensation grout system in the first 120 m of the tunnel drive and permeation grouting for a 30-m length of tunnel beyond to protect a building and a 4-m-wide storm drain. Hayward Baker used horizontal directional drilling (HDD) to install horizontal compensation grout pipes extending 62 to 127 m from the braced excavation, 2 m above the crown of the twin tunnels. Drilling of the pipes produced no measureable surface settlement and pre-conditioning of the ground through the tube-à-manchette (TAM) pipes prior to tunneling was controlled to heaves less than 4 mm.

3 >> PRESSURIZED TUNNELING

It was recognized that settlements during tunneling could be controlled without compensation grouting if the ground control capabilities of the EPBM were fully mobilized at start-up. This proved to be the case. Surface settlements throughout were well below the 6-mm surface settlement level planned for initiation of compensation grouting as the TBM passed.

Recognizing the shallow depth of the tunnel and the limited tunnel distance to monitor ground movements and confirm that TBM ground control procedures were in place before passing beneath structures, the contract required a “tunneling performance demonstration zone” that was approximately 18 m of tunnel construction prior to starting under the structures. This zone is actually an unexcavated berm below the deck beams for the station excavation.

RCC prepared a plan for sequential pressurization of the cutterhead, and the overcut and tail gaps as they passed through the portal seals. Bentonite was injected into the overcut gap between the 1.5-m advances and a two-component grout was injected through the tail skin as the shield was advanced.

Limited surface access for installing and monitoring extensometers and settlement points led to the installation of horizontal inclinometers in horizontal directionally drilled (HDD) holes extending 117 m from the launch shaft. The inclinometers were a Shaped Accelerometer Array, SAA, consisting of a string of measuring points located at 1-m spacing. They were located 1 m above the tunnel and below the compensation grout pipes so that they measured ground displacements around the advancing TBM, largely unaffected by the stiffness of the overlying compensation grout pipes and the pre-conditioning grout.

During monitoring of the horizontal inclinometers, adjustments were made to correct for drift of the gauges, and each point in the array was zeroed 8 m ahead of the advancing TBM in order to show only the displacements due to tunneling.

Settlement of the horizontal inclinometer points compared closely with the settlement of the deep anchors of extensometer 1 (MPBX 1), located just beyond the berm in the braced excavation and adjacent to the parking structure where tunnel depth was 6 m. A total settlement of 9 mm occurred over the shield (Figure 18). After the shield passed beyond the berm, the settlement 1 m above the tunnel was less than 5 mm as shown by the continuous record provided by the horizontal inclinometer (Figure 19).

Initially, as the TBM cutterhead advanced beneath the berm in the shaft, upper face pressures, and shield overcut pressures as well, increased from 0.4 to 0.7 bars, which were close to the overburden pressure applied by the total height of the 3- to 3.5-m berm and sandbags. The 0.7-bar pressures were maintained until the body of the shield had passed beyond the berm and shaft, and MPBX 1. As a result the first three horizontal inclinometer points, located beneath the berm showed heave but there was no blow-out of muck or bentonite (Figure 19).

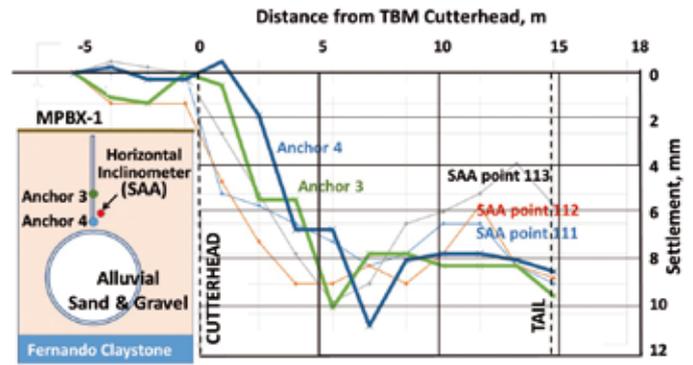


Figure 18 : Settlement above EPBM at start of first drive.

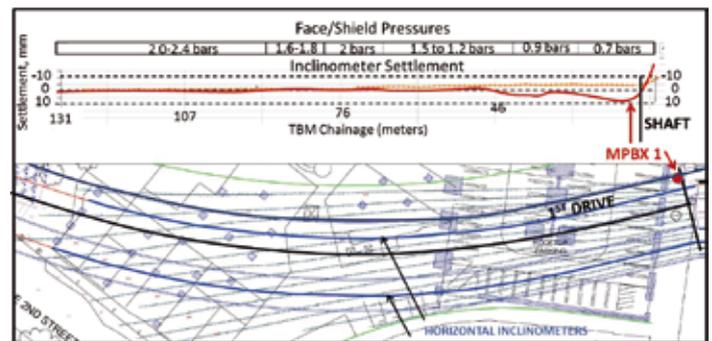


Figure 19 : Horizontal Inclinometer displacement, 1 m above EPBM, First Drive.

Beyond the shaft and berm, tunnel depth was 6 m and overburden pressure was 1.2 bars, higher than the 0.7-bar face/shield pressure. Thus settlement 1 m above the shield was 10 mm.

Once the shield was beyond the shaft wall and the shallow berm, upper face/shield pressures were increased to 1 bar, and then incrementally increased from 1 to 2.5 bars as the depth of the tunnel increased from 6 to 12 m. As a result settlement of the horizontal inclinometer located 1 m above the tunnel decreased to less than 5 mm and then to near zero levels (Figure 19).

Beyond the first 150 m of the tunnel drive, the hard clay/silt (Fernando Formation) occupied the full face of the tunnel. Pressure gauges on the perimeter of the shield showed that pressures were tracking with the face pressures indicating that the conditioned clayey, silty muck was flowing around the cutterhead and filling the overcut gap. The tunnel passed 1.5 m below existing Metro Red Line tunnels with settlements measured by seven extensometers immediately above the tunnel that ranged from 0 to 6 mm after driving both tunnels.

4 >> LARGE DIAMETER PRESSURIZED TBMS AND THE ALASKAN WAY VIADUCT REPLACEMENT TUNNEL

4.1 INTRODUCTION

Marc Isambard Brunel not only built the first subaqueous shield tunnel, but it was also large, even by today's standards. It was a box structure with a width of 12 m to accommodate a twin carriageway with masonry side and center walls. It was unprecedented, innovative and risky.

New records are being set for the size of shielded tunnel boring machines; the largest shielded TBM is now 17.6 m. (However that only represents an average increase in width of 31 mm/year over the 175 years since Brunel completed the Thames tunnel!) Although it will always be difficult to match the unprecedented conditions encountered and the innovative measures used by Brunel, projects with large diameter TBMs have dealt with new challenges, have developed innovative procedures for mitigating risk, and have demonstrated the capability to control the ground to levels far below those that could cause damaging settlement. Following are ground control measures developed and used for urban tunneling with large-diameter earth pressure balance TBMs. The experiences on these and other large-diameter tunnel projects, were an essential part of the design and construction planning and risk assessment for the Alaskan Way Viaduct Replacement Tunnel.

4.1.1 Porto Metro, Portugal: Mechanized tunneling in urban areas

On the Porto Metro, Portugal, a sinkhole and building collapse led to review and implementation of recommendations for coordinated effort and measures for control of the 9-m EPBMs.

The investigation and lessons learned are described in Diez and Williams, 2003, in Raleigh, P., 2006, and in Guglielmetti, V., Grasso, P., Mahtab, A., Xu, S., 2008.

Measures included:

- Development of a Protocol for Advancing the Tunnel (PAT) involving
 - Rigorous management control and monitoring of the TBM operation, combined with analysis of expected tunneling conditions and verification of ground structure behavior by geotechnical monitoring
- Key machine operating targets provided to the tunnel operators
- Project engineering and construction management more involved in the tunneling process.
- Requirement for closed face, pressurized tunneling throughout entire alignment, in the unweathered granite as well as in the variable and weathered, soil-like granite.
- Automatic injection of bentonite into the cutterhead chamber to maintain pressures between advances, and bentonite injection to smear the tunnel face before daily interventions to change cutters in the abrasive granite.
- Primary grouting of the lining through the tail skin. Secondary grouting through segments to check for volume loss
- Filling of the overcut gap with bentonite.

4.1.2 Barcelona Line 9

Barcelona's Line 9 was driven with 9-m EPBMs for double track sections and 12-m EPBMs for double track with station platforms in the tunnel. EPB shield control parameters with target, warning, and alert levels were provided for tunnel reaches and jointly monitored by Contractor and Engineer field representatives. The ability to fill the overcut gap with the conditioned muck flowing from the cutterhead was not considered a systematic ground support measure, and EPB shields were equipped with a dedicated system to execute volume-controlled slurry bentonite injection in the annular gap around the shield (Bono, 2008). On at least one of the Line 9 contracts, a combination bentonite/lime slurry was injected in the shield gap (Escoda, 2017).

4.2 THE ALASKAN WAY VIADUCT REPLACEMENT TUNNEL, SEATTLE: 2017

Washington State Route 99 extends along Seattle's Elliot Bay waterfront on the double deck Alaskan Way Viaduct, a reinforced concrete structure built in 1950 on piles extending through 10 m of hydraulically placed fill and recent alluvium into glacially overridden soils. The structure will be replaced by a single, 2.8-km-long, 17.5-m-diameter tunnel accommodating a double deck structure, each with two traffic lanes and a breakdown lane, with longitudinal ventilation ducts, and emergency passenger egress at the sides. The design-build contractor, Seattle Tunnel Partners (STP), a Joint Venture of Dragados USA and Tutor Perini, selected a Hitachi-Zosen earth pressure balance machine (EPBM).

Descriptions of the tunnel operation and ground behavior are included in papers by Cording, et. al. 2015, 2017, Escoda, et. al. 2017, Fernandez, et. al. 2017, and Mosavat and Mooney, 2015.

The TBM holed through in April, 2017. The double-deck roadway structure was being built as the tunnel advanced and was completed in 2018.

At tunnel depth beneath the Alaskan Way Viaduct are glacially overridden mixed soils consisting of outwash sands and lacustrine clays. At tunnel depth beneath Pioneer Square are hard lacustrine clays, underlain by a continuous, pro-glacial outwash sand and gravel at invert level. To the north, the sand and gravel rises into the tunnel face and dense granular tills and till-like soils are present in the tunnel face and crown (Figure 20a).

The granular pro-glacial outwash had two major effects. In Pioneer Square, its presence at or near invert level below the lacustrine clays required face pressures that balanced the 5-bar water pressure at the invert. Over the length of the tunnel, its high hydraulic conductivity and continuity resulted in a flat groundwater table near high tide level, even though the ground surface increased to elevation +50 m to the north. At its low point, the tunnel crown dropped 30 m below the groundwater level, thus the maximum groundwater pressures were approximately 3 bars at the crown and 5 bars at the invert of the TBM.

4 >> LARGE DIAMETER PRESSURIZED TBMS AND THE ALASKAN WAY VIADUCT REPLACEMENT TUNNEL

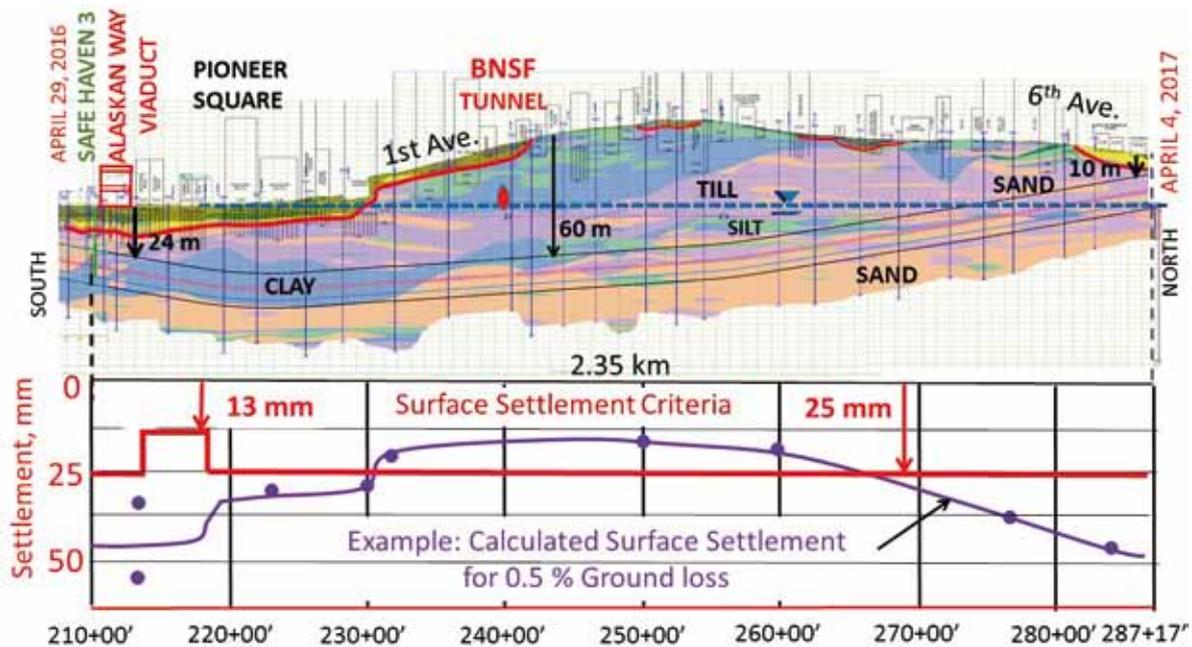
4.3 ASSUMED VOLUME LOSS VS ACTUAL SURFACE SETTLEMENT

Ground loss is commonly reported as a percentage of the volume of the settlement trough with respect to the volume of the tunnel per unit length. The relationship is used to summarize and compare ground control achieved on projects but often information on the causes of the volume loss are not provided so that the relationship between percent ground loss and TBM parameters required to control the ground loss are not known.

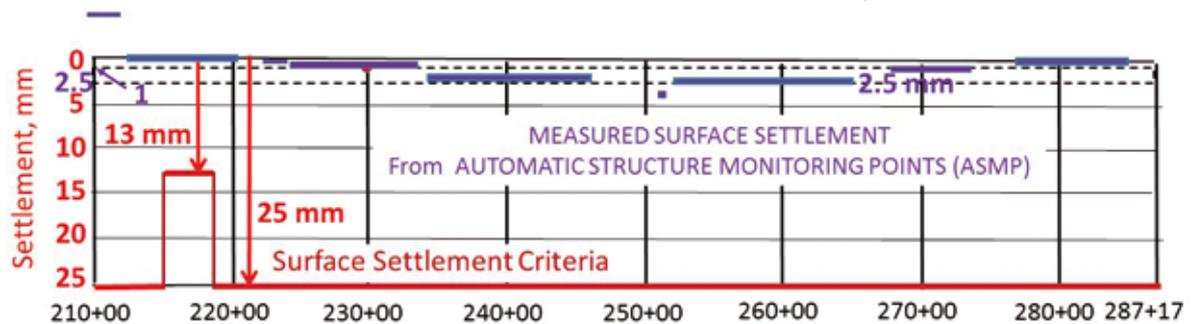
Using a percentage of ground loss obtained from experience on small-diameter tunnels will tend to overestimate the ground loss for a larger tunnel. For example, the gaps around a large-diameter tunnel have dimensions close to those used for smaller diameter tunnels, so the volume of ground lost into an unfilled gap will increase more closely in proportion to the diameter rather than the square of the diameter, which is the relationship for the percent ground loss

For illustrative purposes, Figure 20a shows the surface settlements along the length of the Alaskan Way Viaduct Replacement tunnel for a volume of the surface settlement trough of 0.5%, a value that was used in one of the estimates during preliminary design. In the figure, the surface settlements are determined for a trough width based on a vertical angle of 40 degrees from the springline to the half width, w , of the settlement trough. These calculated settlements exceeded the 25-mm criterion at both the shallower north and south ends of the tunnel.

In contrast to Figure 20a, the actual surface settlements during the passage of the TBM were smallest and not measurable when the tunnel was shallowest (crown less than 30 m deep). They were in the range of 2.5 mm when the tunnel was deepest (crown 45 to 60 m deep), a value that was still an order of magnitude less than the maximum settlement criterion (Figure 20b).



a. Surface settlement criteria and calculated surface settlement for an assumed ground loss of 0.5%.



b. Observed surface settlement.

Figure 20 : Surface settlement calculated from percent ground loss compared to observed settlement on AWRV Project.

4 >> LARGE DIAMETER PRESSURIZED TBMS AND THE ALASKAN WAY VIADUCT REPLACEMENT TUNNEL

The observed surface settlements were based on the automatic structure monitoring points (ASMP) (total station) for structures located above the tunnel. The ASMPs showed variations over a period of a few days or weeks that were greater than the displacements due to tunneling, but the continuous time plots made it possible to compare points above the advancing tunnel with those further away in order to separate out the displacement due to tunneling. The settlements over most of the alignment were too small to be precisely determined from the level surveys.

The results summarized in the following sections provide a perspective on why the constant ground loss assumption did not model the observed ground movements.

4.4 INNOVATION AND RISK

Tunneling for replacement of the Viaduct almost did not happen due to political and economic pressures. A major earthquake in 2001 that damaged the Alaskan Way Viaduct and Seawall led to the realization that the Viaduct had to be replaced.

A tunnel was briefly considered in 2001 but the options brought forward were to replace the viaduct or travel by surface streets. The community, recognizing the disruption and traffic impacts, campaigned for tunneling. A coalition of 300 organizations was formed and proactively engaged and educated state and city officials early in the planning process (Donegan, 2017). Washington State Department of Transportation (WSDOT) re-examined the tunneling option. Twin tunnels driven on separate streets were considered, but ultimately a single large-diameter tunnel was specified.

Dick Robbins, in his 2013 Muir Wood Lecture described the Alaskan Way Viaduct Replacement Tunnel with the heading “MODERN INNOVATION AT WORK” and noted: “Despite all of the current challenges for creative development work in the industry, examples can still be found where innovation is encouraged particularly where project conditions are exceedingly difficult or unprecedented. The SR 99 Viaduct Replacement Tunnel in Seattle, Washington, USA is an example of both modern-day risk sharing and innovative design work.”

To mitigate such risks, a series of preliminary design decisions and contract requirements were prepared. Of particular concern were conditions that would have a different or even unique effect on a larger-diameter tunnel. Risks of tunneling settlement and how the tunnel construction contract was established are described in Hansmire et al. (2011). The tunnel contract had explicit requirements for geotechnical instrumentation and a Construction Monitoring Task Force (CMTF) that were implemented by the design-build contractor, STP.

Figure 21 summarizes ground improvement measures considered and used to mitigate risk by WSDOT and by STP. The specified compensation grouting beneath two buildings in Pioneer Square was eliminated based on the evidence from the SESMP test sections showing that surface settlements were well below the 13 mm criterion for compensation grouting. However, building repairs were needed. Large differential settlements and distortions and cracking had occurred over the past century in the two structures, which were on timber pile foundations extending through the loose hydraulic sand fill and recent alluvial deposits. STP stabilized and stiffened the foundations, in one building with micro piles and in the other with grade beams.

The primary ground control measure, in both WSDOT’s contract requirements and STP’s tunneling plan and operation, was to monitor and control the TBM operation. Measures included:

- Overcut gap pressurized and filled with bentonite.
- Earth pressure gauges on the shield body to monitor pressures in the overcut gap.
- Closely spaced extensometers, at an average of 16 m centers, over the entire alignment.
- Piezometers added in each of the extensometer borings.

4.4.1 South End Settlement Monitoring Plan

The South End Settlement Monitoring Plan (SESMP) section, was located beneath the project construction yard and immediately adjacent to the Alaskan Way Viaduct. STP proposed extending project contract limits further south to have additional distance for controlling and monitoring TBM performance and gaining tunnel depth prior to passing beneath the Viaduct.

Monitoring of ground movements and TBM performance in the first 300 m of the SESMP section in November 2013 showed that settlements were controlled to small values as the tunnel crown passed into the glacial soils (Cording et al., 2015).

Tunneling was halted in December 2013 and an access shaft was installed to remove and repair the cutterhead and main bearing. In January 2016, the reassembled TBM was advanced out of the shaft through the last 100 m of the SESMP section to the hold point at Safe Haven 3. TBM operations and observations in two test sections confirmed that ground control measures were in place and that settlements were being controlled as the TBM approached Safe Haven 3. A compressed air intervention was conducted once the cutterhead was in the jet-grouted Safe Haven. The advance was continued beyond Safe Haven 3 on April 29, 2016

4 >> LARGE DIAMETER PRESSURIZED TBMS AND THE ALASKAN WAY VIADUCT REPLACEMENT TUNNEL

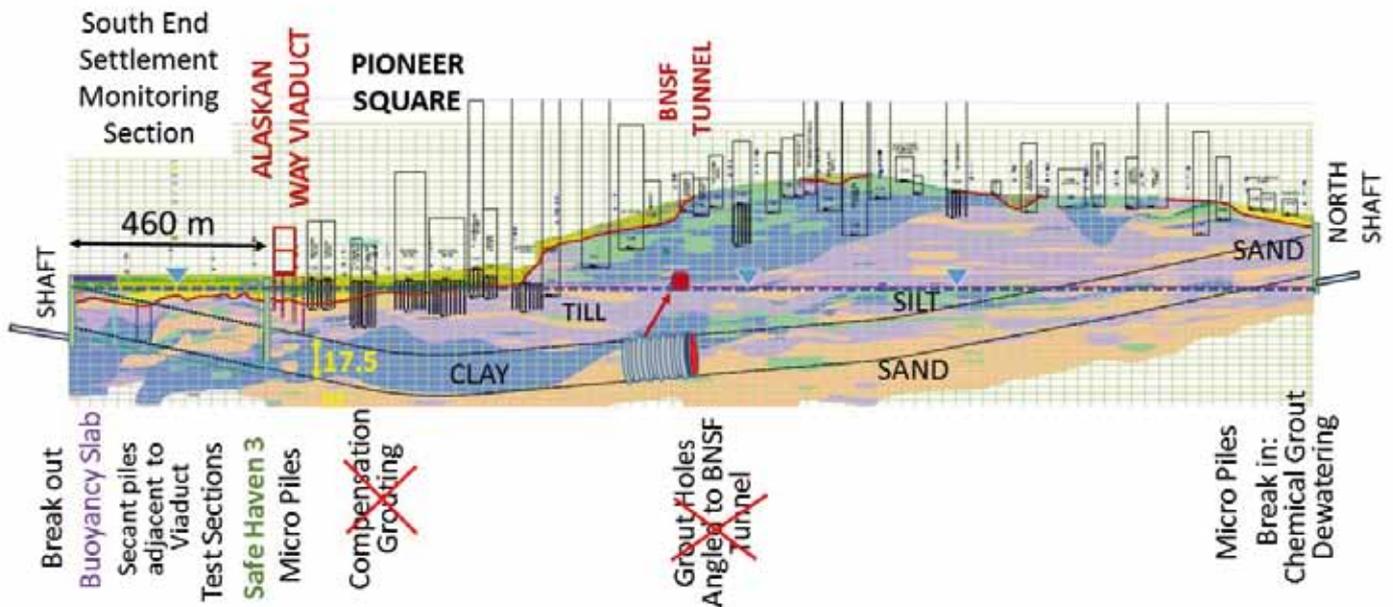


Figure 21 : Profile of Alaskan Way Viaduct Replacement Tunnel Alignment and ground control measures.

4.5 COORDINATION PLAN

Prior to advancing beyond the hold point at Safe Haven 3 and beneath the viaduct, a coordinated program of monitoring of ground and TBM performance and control, review, and adjustment of the TBM operation became fully organized that drew together the capabilities of the STP managers, TBM engineers, operators, quality control, safety, geotechnical monitoring teams and WSDOT engineering and construction management (CM) (Figure 22). An environment was created in which information was shared and information-based decisions were timely executed.

4.5.1 Construction Monitoring Task Force (CMTF)

The Task Force effort included:

- Daily meeting with STP and WSDOT teams, chaired by tunnel manager, to review previous day's operation.
- Review of Daily Tunnel Parameter Log to be used by TBM operators showing target and green range for key operating parameters affecting ground control (Figure 23).

4.5.2 Coordination of the TBM operation



Figure 22 : Coordination among owner, contractor, tunnel operation and ground monitoring.

4 >> LARGE DIAMETER PRESSURIZED TBMS AND THE ALASKAN WAY VIADUCT REPLACEMENT TUNNEL

Daily TBM Parameter Log		Report #:	135	revision #:	1	
		Start Station:	25838	Current Ring #:	977	
		Date From:	12/9/16	Time From:	5:00 AM	
		Date To:	12/10/16	Time To:	4:59 AM	
TBM PARAMETERS	Target Advance Speed (mm/min)	40	Target Total CHD Force (kN)	45,000		
	Target CHD Torque (kN)	40%	Target Total Thrust Force (kN)	355,000		
	Target CHD Rotation Speed (rev/min)	1.0	Penetration Rate (mm/rev)	40		
WORKING PRESSURE	TOP Target Earth Pressure (bar): (average of sensors 11 & 12)	3.2	Green Range	3	to 3.5	
	Keep System Settings (bar): (based on average of sensors 11 & 12)	lower	3	Upper	3.05	
GROUT PRESSURE	TOP Target Grout Line Pressure (bar)	5.3	Green Range	5	to 5.8	
	MID. TOP Target Grout Line Pressure (bar)	6.1		5.8	to 6.6	
	MID. BOT. Target Grout Line Pressure (bar)	7.1		6.8	to 7.6	
	BOTTOM Target Grout Line Pressure (bar)	7.5		7.2	to 8.0	
GROUT VOLUME	Grout Volume Target (m ³)	26	Green Range	22.1	to 31.2	
GAP	TOP Target Gap Line Pressure (bar)	4.0	Green Range	N/A	to 8.0	
	Gap Volume Target (m ³)	4.14	Green Range	N/A	to 5.4	
	Gap Keep Settings (bar): (based on gap sensors #8 and #9)	Lower	2.80	Upper	2.85	
SOIL CONDITIONING	Planned Ground Conditioning Recipe: Recipe 1: SLF (3.0%) plus Rehosoil 211 (0.5%)					
	PER Target	12	PER Range:	10 to 14	FIR Target: 85% FIR Range: 40% to 100%	
	Slump Range:	5			Concentration Target: 3.0% (Q/N) Range: 5% to 5%	
	Target Ball Scale Weight (kg)	285 Density: 472 804	Comments: Density/Slump tests to be carried out at least once per shift. Except when consistency is too wet that it will be once per ring.			
Coverage (m)	60	FIELD PRESSURE	Number	10285_214_243_99 08 07 25842 08	Pressure Value (bar)	1.45
	Present Soil Unit: (As per GBM)	Muddy ESU 5				

Figure 23 : Example of Daily TBM Parameter Log; 12/9/2016 - 12/10/2016.

4.5.3 TBM ground control operation

The diameter of the TBM was large enough that the spokes were designed to free-air access to the cutterhead for changing rippers and disk cutters (Figure 24). Disk cutters on alternate spokes prior to launch were only used for a short distance in the SESMP section and were replaced with additional rippers. The cutterhead and ribbon screw conveyor were designed to accommodate 900-mm boulders. Scrapers on the edge of the spokes were replaced during compressed air interventions. Ports on the shield body shown in Figure 24 were designed for probing over the cutterhead and for bentonite injection into the overcut gap, but were only used for the latter.

Recipes for conditioning the muck include anti-clay polymer and foam for the clays. For the sandy soils, polymer, foam and bentonite were used. Up to two tons of bentonite were added to the injected slurry per 2-meter advance.

Figure 25 is an example of upper and lower face pressures and upper shield body pressure measured over a two-week period. There were 13 earth pressure cells in the chamber for monitoring face pressure and 6 earth pressure cells on the upper arch at the front and middle sections of the shield body for measuring pressures in the overcut gap.

The upper pair of face pressure cells served as the reference for controlling face pressures. Between advances, bentonite was automatically injected into the chamber to maintain the upper face pressure at a constant 0.2

bar below target pressures. The increase in lower face pressure between advances resulted from an increase in density of the conditioned muck as bentonite slurry replaced air coming out of the foam. A vent line to the top of the chamber was used for automatic purging and venting of air onto the conveyor belt.

Bentonite slurry injected into the overcut gap was volume controlled and automatically injected throughout the advance to fill the 30-mm gap. Pressure in the gap tracked with the face pressure. Between advances, pressure in the overcut dropped toward groundwater pressure, and bentonite was injected to maintain pressures above the groundwater pressures.



Figure 24 : EPBM cutter head in launch shaft, Alaskan Way Viaduct Replacement Tunnel.

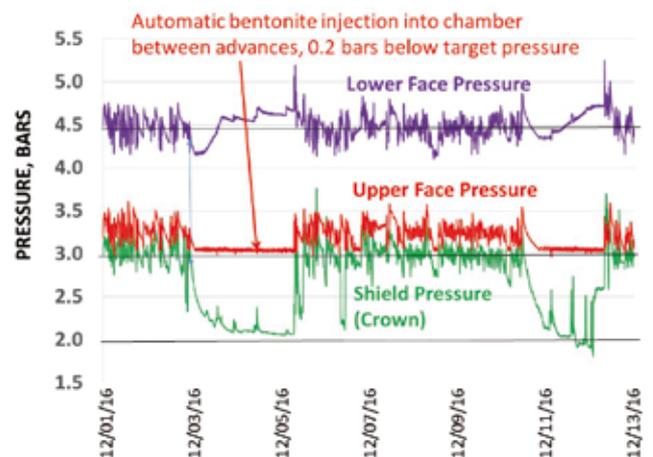


Figure 25 : Example of TBM face and shield pressures, 12/1/2016 - 12/13/2016, Station 255+50 to 260+00.

4 >> LARGE DIAMETER PRESSURIZED TBMS AND THE ALASKAN WAY VIADUCT REPLACEMENT TUNNEL

4.5.4 Geotechnical Monitoring

After every 2-m advance of the TBM, the geotechnical team provided a report on ground conditions and time plots of monitoring data from SolData instrumentation within a 60-m zone around the advancing TBM. The information was provided to the operations team and inspectors in the tunnel and emailed to WSDOT and STP team members (Figure 26).

Monitoring at the Source. Primary emphasis was placed on reporting the ground movements at their source, providing continuous time plots of extensometer displacements and the piezometric pressure immediately above the advancing TBM. Results were correlated with the time plots of face and shield overcut gap pressures.

Monitoring at the Surface. Time plots were provided of surface and structure displacements confirming that settlements were within allowable criteria.

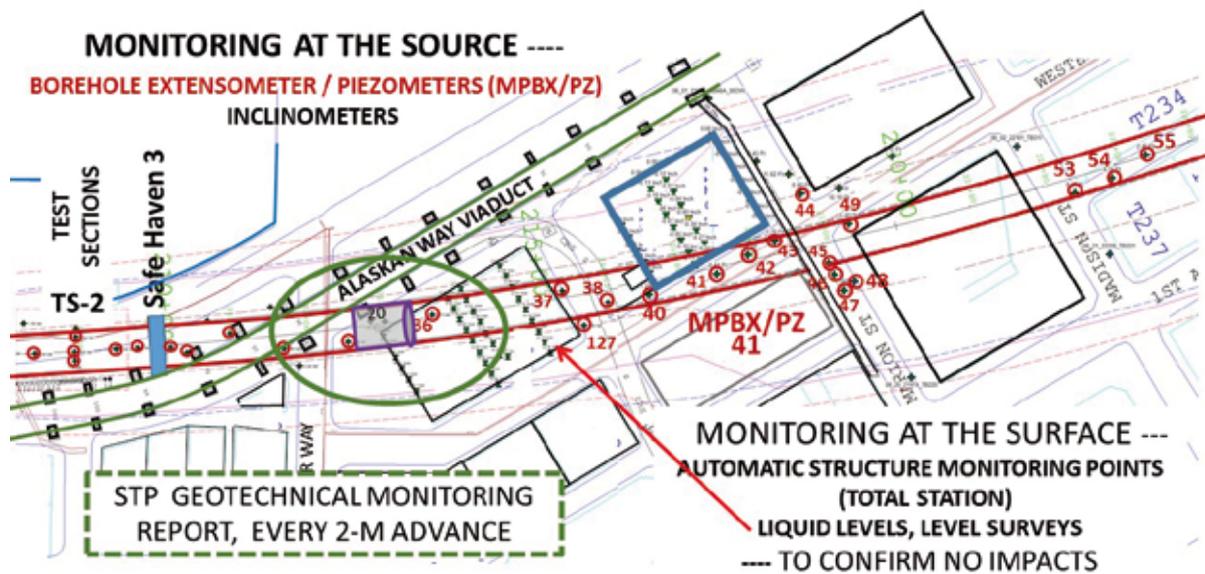


Figure 26 : Ground and structure monitoring from Safe Haven 3 to Alaskan Way Viaduct and Pioneer Square.



Figure 27 : Alaskan Way Viaduct Replacement tunnel alignment under Seattle.

4 >> LARGE DIAMETER PRESSURIZED TBMS AND THE ALASKAN WAY VIADUCT REPLACEMENT TUNNEL

4.6 TBM ADVANCE BENEATH ALASKAN WAY VIADUCT AND PIONEER SQUARE

4.6.1 Alaskan Way Viaduct Crossing

The TBM advanced from Safe Haven 3 at Station 210+00 on April 29, 2016, passing 4 to 6 m beneath pile foundations for four Alaskan Way Viaduct piers (Figure 26 and 27).

The viaduct was shut down by Washington State Department of Transportation for a period of two weeks, but was reopened to Monday rush hour traffic a week early as the TBM cutterhead passed beyond the last viaduct foundation, with no negative effects.

During the passage beneath the Viaduct, pressurization of the overcut gap on the shield was close to overburden pressures causing 5 mm heave of the Viaduct foundation.

The large pressurized area of the shield (a cylinder 17.5 m in diameter and 20 m long) can lift a large area and recover settlements further above the tunnel than is possible with grouting behind the tail, which only pressurizes over the width of the 2-m tunnel lining ring. The ability to recover displacements above the shield at pressures less than overburden pressure was also demonstrated at one of the extensometers in Pioneer Square in sands, as the dynamic groundwater pressures above the shield dropped and shield pressures provided an increased effective pressure that supported the wedge of soil above the shield body, recovering displacement over the shield with no measurable surface settlement.

4.6.2 Displacements and Pressures in Clay beneath Pioneer Square.

During tunneling beneath buildings in Pioneer Square, there was no settlement of structures or the ground surface (as measured by the ASMP points on the structures, the liquid level lines within the structures, and the survey leveling).

Most of the piezometers located 1.5 m above the crown in this section were in clay. The undrained behavior of the clay resulted in a reduction in porewater pressure over the shield due to the difference between the overburden pressures and the pressures around the shield.

Figure 28 illustrates the response that typically occurred throughout Pioneer Square when clay was in the tunnel crown. Piezometric pressure in PZ 41 began dropping to a value 1 bar below ambient pressure over the front portion of the shield and recovered over the back half of the shield and tail. At the same time, the deep anchor, 1.5 m above the shield, displaced downward approximately 1 mm.

Inclinometers placed ahead of the TBM recorded small lateral displacements toward the cutterhead. Inclinometer 53 showed 2 mm when the TBM approached within 3 m of the inclinometer (Figure 29).

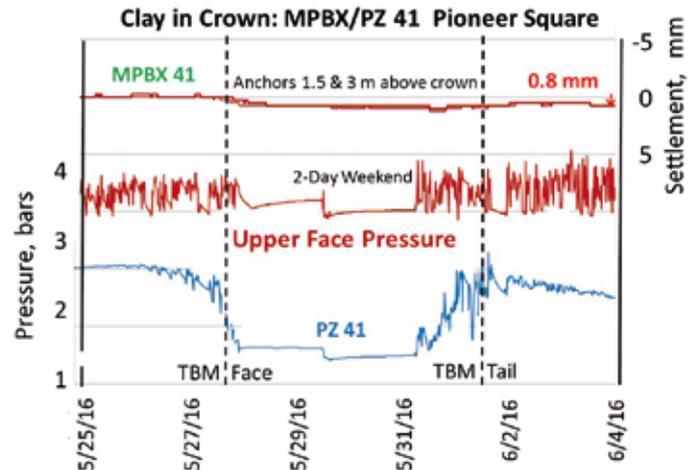


Figure 28 : Settlement and pressures in clay above advancing TBM, Pioneer Square.

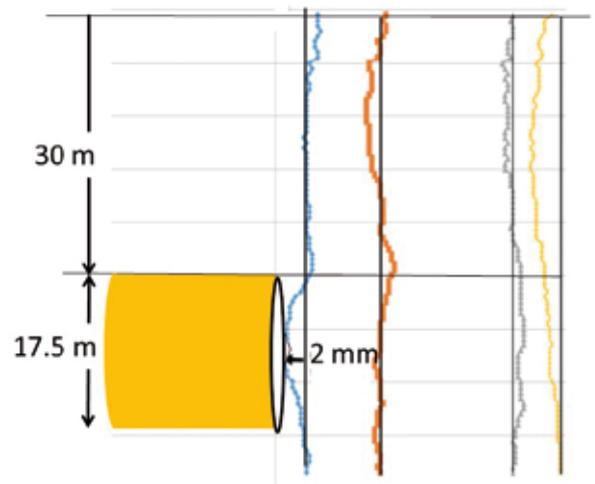


Figure 29 : Horizontal displacement toward TBM face Inclinometer # 53, Pioneer Square .

4 >> LARGE DIAMETER PRESSURIZED TBMS AND THE ALASKAN WAY VIADUCT REPLACEMENT TUNNEL

4.7 TBM ADVANCE WITH INCREASING DEPTH TO BNSF TUNNEL

North of Pioneer Square along First Avenue, cover over the tunnel increased to 60 m while groundwater levels remained flat at elevation +1.5 to 3 m, near tide level in Puget Sound. With the increasing cover, the differential pressure between overburden pressure and the face/shield pressures increased (12 bars overburden vs 4 bars face pressure). Approaching the BNSF tunnel (Figure 30) the higher differential between the overburden pressures and TBM pressures resulted in deep anchor settlement between 8 to 15 mm (Figure 31).

Surface settlements of 2.5 mm is added to the differential settlement of the deep extensometer anchors to obtain the total settlements shown in the profile, Figure 31, and time plots in Figures 32 and 33.



Figure 30 : TBM crossing at BNSF Tunnel and Pike Street Adit.

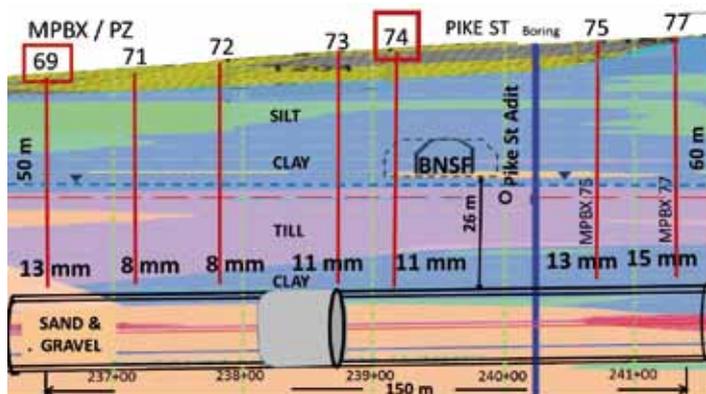


Figure 31 : Settlement 1.5 m above TBM approaching BNSF tunnel.

4.7.1 Pressures and displacements in Clay at 55-m-depth

Extensometer/piezometer, MPBX/PZ 74, in clay at 55 m depth (Figure 32) shows similar behavior to that observed at shallower depth in Pioneer Square (MPBX/PZ 41, Figure 28). However, because of the greater

differential between overburden pressure and the face/shield pressures (12 bars versus 4 bars), the porewater pressures in the clay began dropping 15 m ahead of the shield, rather than over the shield, and settlements over the shield were higher.

4.7.2 Pressures and displacements In Sand at 50-m-depth

Settlements and piezometric pressures at MPBX/PZ 69, in sands at and above the tunnel crown, are shown in Figure 33. Groundwater pressures 1.5 m above the shield were 2.2 bars and began increasing 35 m ahead of the TBM, to values temporarily approaching within 0.6 bars of the 4-bar upper face and shield pressure. Deep anchor total settlement of 13 mm occurred over the shield, the same range as those measured in the clayey soils in the high cover section.

For a hydrostatic distribution of pressures, piezometers 1.5 m above the shield would be 0.15 bars below the shield crown pressure. The piezometric pressure must be at least 0.3 bars below the shield pressure in order for the soil in the 1.5 m interval to be supported. As shown in Figure 30, the hydraulic gradient between the bentonite slurry and the piezometer was high enough to create a differential pressure of 0.6 bars. Over a four-day period between advances, penetration and caking of the bentonite slurry increased the hydraulic gradient and piezometric pressure dropped to 1 bar below the shield pressure in the overcut gap.

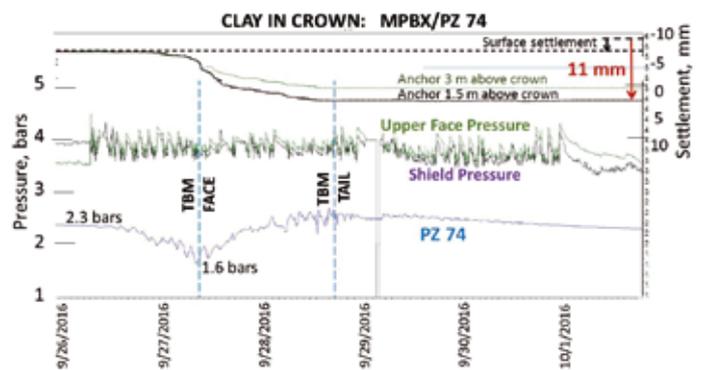


Figure 32 : Extensometer displacements and piezometric pressures above TBM in Clay, 55 m depth.

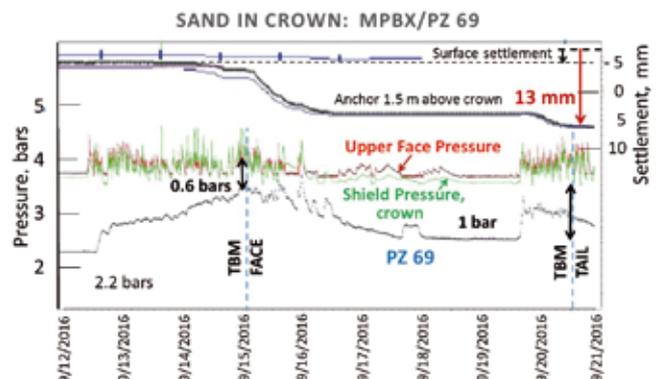


Figure 33 : Extensometer displacements and piezometric pressures above advancing TBM in sand, 50 m depth.

4 >> LARGE DIAMETER PRESSURIZED TBMS AND THE ALASKAN WAY VIADUCT REPLACEMENT TUNNEL

4.7.3 Settlement distribution in BNSF Tunnel and Pike Street Adit

The TBM passes at a skewed angle beneath the BNSF tunnel. Automatic structure monitoring points (ASMPs) precise survey prisms) were installed on the walls and floor of the BNSF tunnel.

Small settlement of 0.5 mm began to be observed in the BNSF tunnel as the TBM approached within approximately two diameters (35m) of the BNSF tunnel and were skewed to the east (right) due to the angle of TBM approach. After the TBM passed the BNSF tunnel, total settlements reached 5 mm at the TBM centerline (Figure 34).

The precision of the instrumentation in both the BNSF tunnel and the Pike Street Adit permitted the measurement of the shape of a classic Gaussian settlement trough despite the small settlement magnitude.

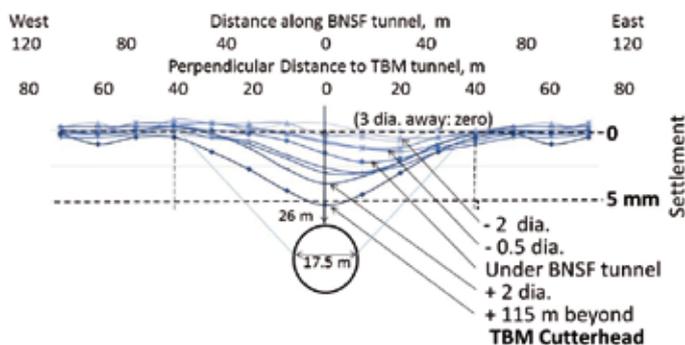


Figure 34 : Settlement distribution measured with ASMP prisms in BNSF tunnel.

The Pike St. Adit is immediately beyond the BNSF crossing and 22 m above the crown of the AWRV tunnel. Settlement within the adit was measured with a tilt beam (essentially a horizontal in-place inclinometer). The settlement trough was similar to that observed in the BNSF tunnel. (Figures 35 and 36).

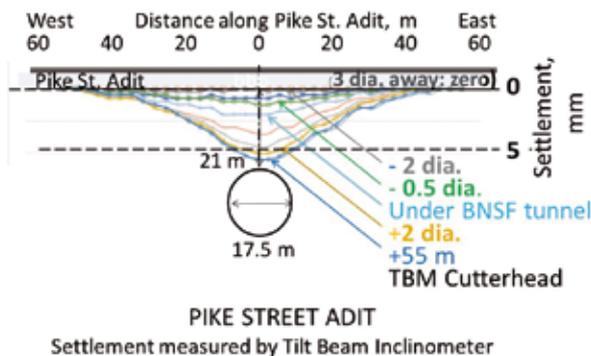


Figure 35 : Settlement distribution measured with tilt beam (horizontal inclinometer) in Pike Street Adit.

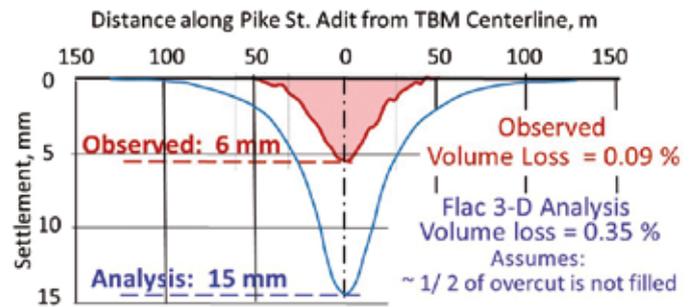


Figure 36 : Observed settlements compared with pre-construction analysis, Pike Street Adit.

Prior to tunneling, three-dimensional FLAC analyses were conducted to assess the effect of the TBM passage on the BNSF tunnel and the Pike St Adit. In the analysis, the ground loss was assumed to be one half of the volume of the overcut gap, which resulted in an estimated settlement of 15 mm (Figure 36). This conservative estimate showed that distortions would be below damage levels in both the adit and BNSF tunnel. Although the analysis used a small-strain constitutive model, it showed a wider distribution of settlement than the observed settlement trough.

Measurements as the BNSF Tunnel was approached indicated that settlement volumes and displacements would be smaller than those assumed in the analysis, because the overcut gap was filled.

4.8 SETTLEMENT IN SANDS AT NORTH END OF TUNNEL

Beyond the BNSF crossing, granular soils were increasingly concentrated at and above the tunnel face, consisting principally of lacustrine silts and outwash sand and gravel and sandy till-like soils including sand and gravel with less than 5% fines.

Polymer was added to the bentonite injected in the shield gap in order to increase viscosity and limit penetration in the sandy soil. As a result the outward hydraulic gradient, was sufficient to allow the pressures in the shield gap to support groundwater pressures and the effective pressure from the wedge of soil above the 17.5-m-width and 20-m length of the shield.

The monitoring results confirmed the effectiveness of pressurizing and filling of the overcut gap. Settlements 1.5 m above the shield were the same for both sandy and clayey soils over the entire tunnel drive. They were less than a millimeter when the differential between overburden and face/shield pressures was less than 2 bars in both the clays at the south end and the sands at the north end (Figure 37).

4 >> LARGE DIAMETER PRESSURIZED TBMS AND THE ALASKAN WAY VIADUCT REPLACEMENT TUNNEL

4.9 SETTLEMENT DUE TO DIFFERENTIAL PRESSURE

In Figure 38, the deep anchor settlements are plotted in the opposite direction, to illustrate their correspondence with difference between overburden and face/shield pressures. Differential pressures were particularly high at depths of 60 m because the groundwater pressures and required face/shield pressures were low. Even with the differential of 8 bars, the high stiffness of the glacially-overridden soils resulted in relatively small deep anchor settlements of 8 to 11 mm (a volume of less than 0.1%), not due to ground loss but due to elastic displacement.

At the shallower depths at the north and south ends, for differential pressures less than 2 bars, deep anchor settlements were 1 mm and volume loss was on the order of 0.01 %. The small displacements appear to be due to higher stiffness of the ground at small strains. The settlements were a function of the difference between overburden pressures and face/shield body pressures and inversely proportional to the stiffness, Young's modulus, E , of the ground.

It should be noted that in more compressible soils, once the gaps are pressurized and filled, settlements can be reduced, as needed, by increasing TBM face/shield pressures above the minimum values required to balance ground water and effective active pressures.

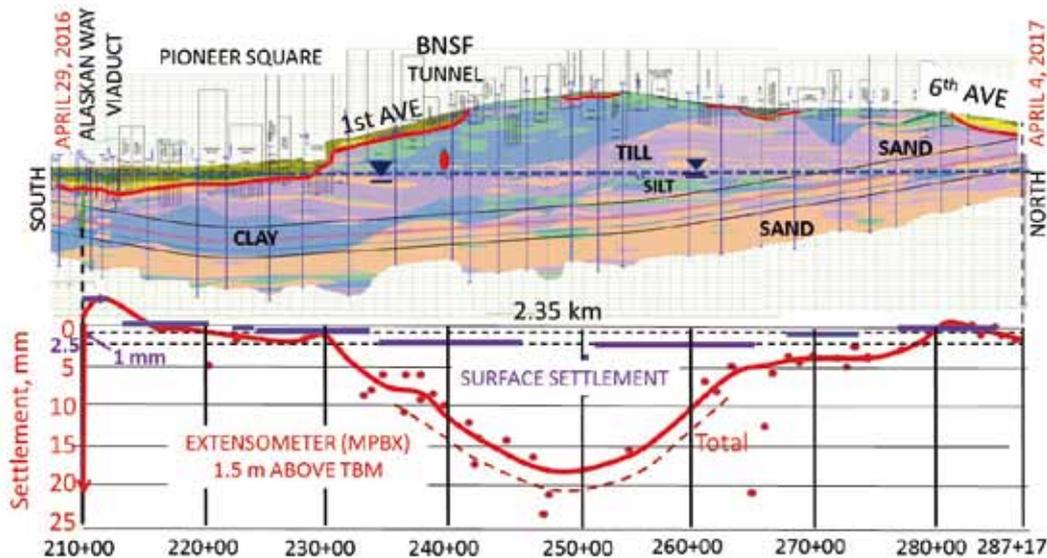


Figure 37 : Observed settlements at the surface and 1.5 m above TBM.

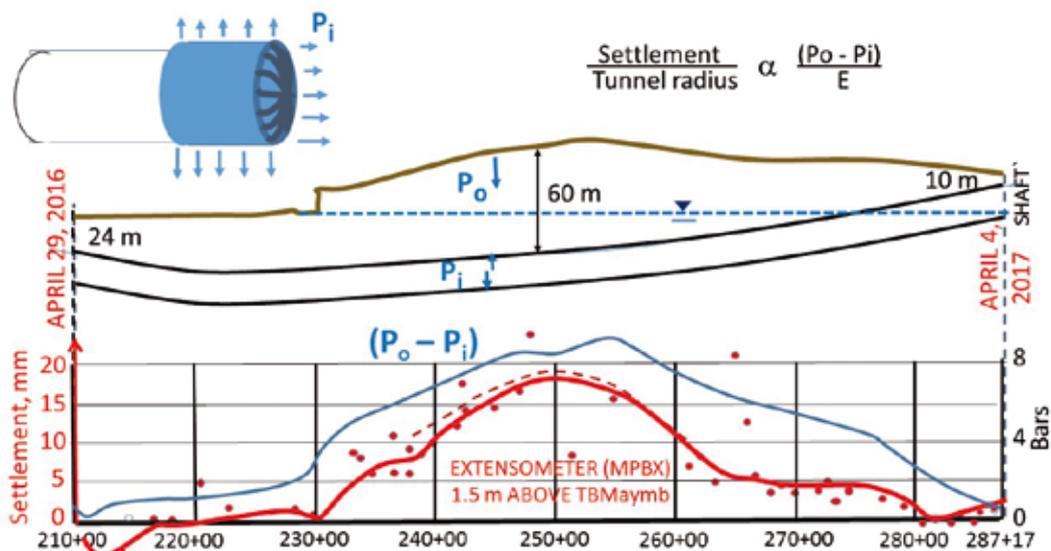


Figure 38 : Settlements above TBM compared with overburden minus face + shield pressures.

5 >> CONCLUSIONS

We are witnessing a revolution, and it is not over. Progress in pressurized TBM operation has reached the point where consistent control of ground movements is being achieved, preventing damaging ground movements.

Consistently controlling pressures and minimizing settlement throughout the tunnel drive serve as a demonstration to project participants and communities alike that structures along the tunnel alignment will be protected.

COORDINATION

The ground control achieved for EPBM tunneling at shallow depth and for large diameter TBMs has benefited from and depended on a coordinated, collaborative effort between owner and contractor teams. The Protocol for Advancing the Tunnel instituted on the Porto Metro and the Construction Monitoring Task Force in Seattle formalized the process. In these cases, the events and changes on the projects resulted in the parties seeking special, cooperative, interactive efforts.

Reach by reach tunnel logs of key operating parameters were used on the Porto Barcelona line 9, and Toronto Northern Tunnels Projects. Daily tunnel logs used on the Alaskan Way Viaduct project provided targets and green range for key TBM operating parameters and automatic settings. The Construction Monitoring Task Force reviewed the values obtained for each of the ring advances from the previous day.

The pressurized tunnel boring machine employs complex systems, produces a plethora of electronic data, as well as some critical non-electronic observations, and engages a multitude of engineering disciplines, underscoring the importance of a coordinated effort among owner and contractor teams, regardless of the contracting procedures and contract form.

Extensive electronic and non-electronic data is also obtained from monitoring the ground and structures. It is most helpful to have a geotechnical monitoring team embedded within the TBM operation to interpret and correlate the information with key TBM operating parameters.

MONITORING GROUND BEHAVIOR AT THE SOURCE

The cases presented in the paper highlight the importance of monitoring ground behavior at its source in order to understand the causes of ground movement and changes in groundwater pressure and to confirm that control is being achieved with the pressurized TBM operation.

On the Alaskan Way Viaduct Replacement tunnel, monitoring of ground behavior at the source was conducted with combination extensometers/piezometers at an average spacing of 16 m, which provided an almost continuous view of the effect of TBM face and shield gap pressures on ground displacement and changes in groundwater pressures.

Measurements made in both clays and sands, at depths ranging from 10 to 60 meters, confirmed that filling and pressurizing of the gaps was preventing ground loss. The small displacements around the advancing TBM were related to the stress changes due to the differential between overburden and face/shield pressures. Once the gaps are filled, displacements can be reduced further, if needed, by increasing the upper face and shield pressure. The condition would be most likely a concern in soft soils, where there are significant pressure differences. Displacements due to the pressure differential on the Alaskan Way Replacement Project were not measurable when the tunnel was at shallow depth and were at least an order of magnitude below allowable settlements throughout.

Monitoring of the piezometric pressures around the TBM and comparing them to the face/shield gap pressures provided information needed to determine the effectiveness of the face and shield gap conditioning, injections, and pressures in creating a hydraulic gradient sufficient to support the groundwater and the effective active forces in the face and above the shield.

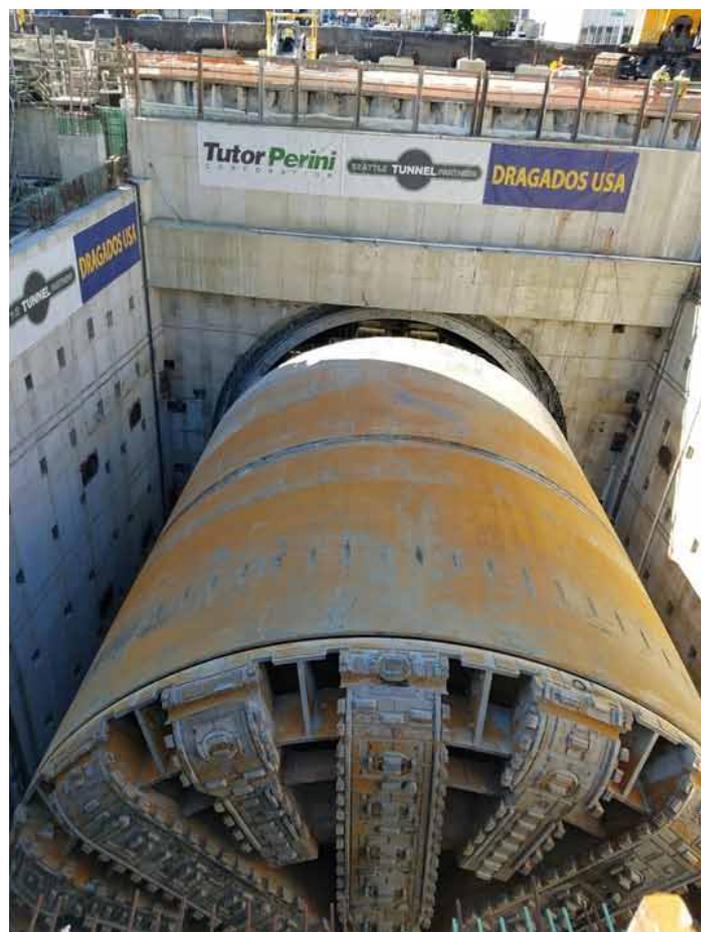


Figure 39 : Alaskan Way Viaduct Replacement EPBM in in exit shaft.

5 >> CONCLUSIONS

PREVENTION OF GROUND LOSS

The pressurized TBM is more than a pressurized face TBM, it has a pressurized envelope that extends from the cutterhead, around the body of the shield to the grout being injected through the tail of the shield to fill the gap around the lining, throughout the tunnel drive. Positive filling and pressurization of gaps prevents ground loss.

Estimates of ground loss percentages based on small diameter shields will result in overestimation of the ground loss for a large diameter shield. For example, the perimeter increases in proportion to the diameter but the gap dimension remains relatively small, so that the gap dimension increases closer to the diameter than the square of the diameter.

Tunneling at shallow depth requires special investigations and special control, but the model that assumes a given percentage of ground loss does not represent the behavior of a well-controlled pressurized TBM operation and can lead to the conclusion that the tunnel should be deeper or smaller or the damage will be moderate to severe and ground improvement procedures are required, rather than focusing on controlling the TBM operation. The ability to control ground movements with the tunneling process reduces the need for such procedures.

At the same time, there are many situations where ground improvement methods are essential to the pressurized TBM operation. They are used on most projects, particularly in conditions where the TBM cannot be fully pressurized. Drilling of multiple holes for ground improvement requires the use of control measures that fit the ground and prevent ground loss. In some cases, the pressurized TBM was far better preventing ground loss than the drilling operation.



Figure 40 : Alaskan Way Viaduct Replacement EPBM in exit shaft.

6 >> ACKNOWLEDGEMENT

I have been privileged to work with individuals and organizations that are committed to excellence in building underground projects and in communicating and advancing the state of the tunneling art. Some of the projects on which we have worked are presented in this lecture. I am particularly indebted to Harvey Parker, William Hansmire, Anthony Stirbys, Hossein Bidhendi, Michael DiPonio, Jason Choi, William Edgerton, Randall Essex, Glen Frank, Brian Fulcher, and Peter Raleigh for their perspectives and contributions on the projects and topics of this lecture, and am most appreciative of the opportunities we have had to work together over the years.

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